VOLUME 83 NO. SM3
JULY 1957

JOURNAL of the

Soil Mechanics and Foundations Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, 3/16-in. high (with all other symbols and characters in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller in the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper within a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11-in. paper within an invisible frame that measures 6½-in. by 10½-in.; the caption should also be included within the frame. Because illustrations will be reduced to 69% of the original size, the capital letters should be 3/16-in. high. Photographs should be submitted as glossy prints in a size that is less than 6½-in. by 10½-in. Explanations and descriptions should be made within the text for each illustration.

8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

This Journal is published quarterly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class mail privileges are authorized at Ann Arbor, Michigan.

EM,SM.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

July, 1957

Papers

DIVISION ACTIVITIES

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

MISSISSIPPI VALLEY GEOLOGY - ITS ENGINEERING SIGNIFICANCE

C. R. Kolb,* and W. G. Shockley,** M. ASCE
(Proc. Paper 1289)

SYNOPSIS

The increasing importance of geologic information to the design of engineering structures in the lower Mississippi River valley is pointed out. The various environments of alluvial deposition and the resulting soil formations are discussed, especially with relation to engineering problems associated with them.

INTRODUCTION

It has been only a little more than a decade since the geology of the lower Mississippi valley has contributed to any significant extent to the various problems which beset engineers working within the area. Indeed, it has been only slightly more than a decade since the geology has been known in sufficient detail to be of practical value to engineers. Much work still needs to be done, many refinements to geologic concepts need to be made, and useful correlations between the various geologic environments and the important physical properties of the soils that they represent are necessary. Nevertheless, the basic outlines of the geology of the valley are known and a knowledge of its implications to foundation, flood control, bank stability, and other problems are mandatory for the practicing engineer working within the area.

The alluvial valley of the lower Mississippi River forms an elongate lowland extending from Cairo, Illinois, to the Gulf of Mexico, covering an area of approximately 50,000 square miles. Its formation is closely associated with the last stages of glaciation, the accompanying fall and the subsequent rise of sea level. The fall in sea level resulted in the scouring of an entrenched valley beneath the present floodplain surface. Rising sea level

Note: Discussion open until December 1, 1957. Paper 1289 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83, No. SM 3, July, 1957.

*Chief, Geology Branch, Soils Div., Waterways Experiment Station, Corps of Engrs., U.S. Dept. of the Army, Vicksburg, Miss.

**Chief, Embankment and Foundation Branch, Soils Div., Waterways Experiment Station, Corps of Engrs., U.S. Dept. of the Army, Vicksburg, Miss.

brought about deposition of sands and gravels within the lower portions of this trench, followed by the deposition of a topstratum of finer-grained material. During an estimated interval of 25,000 years, as sea level rose, the Mississippi gradually changed from an overloaded, shallow stream with a braided regimen to its present deep, single-channel, meandering regimen. Details of the hypothetical sequence of geologic events are well described by Fisk.[1,2]

For the past ten years this sequence of geologic events has been studied in some detail by geologists working for the U. S. Army Corps of Engineers. Studies were conducted of the processes of alluvial and deltaic sedimentation, the various environments of alluvial deposition, the resulting soil textures and stratum thicknesses and continuities, and the physical properties of the soils deposited within the various environments. Thousands of borings made within the alluvial valley for engineering and other projects have been examined. Of immeasurable importance has been the gradual refinement of air photo techniques of interpreting the various depositional environments. One of the more obvious purposes of these studies, carried on as time and funds have permitted, was the accurate prediction of engineering soil types without extensive and expensive boring programs.

The deposits which fill the entrenched valley of the Mississippi and make up its alluvial plain can be divided into two major groups: the sand and gravel substratum and a finer-grained topstratum. As shown in Figure 1, the topstratum can be further subdivided into four general types: (a) ancient braided stream deposits of the Mississippi River and its tributaries, (b) meander belt deposits, (c) backswamp deposits, and (d) deltaic plain deposits. The discussion which follows summarizes the method of deposition, the resulting soil sequence, the physical properties of each of these groups, and mentions some of the engineering problems associated with each group.

The Substratum

The substratum consists of a wedge of coarse-grained material laid down during the earlier stages of the filling of the entrenched valley of the Mississippi River. It consists predominantly of clean sand and gravel with the coarsest material normally found at depth. Cobbles up to 4 inches in diameter are sometimes encountered near the base of the unit. Occasional lenses of clay, sandy silt, or silty sand are also found, but these are rare and discontinuous. Thicknesses of the substratum and the depth to the top of the substratum generally increase in a down-valley direction. The substratum is often encountered at depths as shallow as 10 feet and may average only 50 feet thick in the northern part of the valley. Depth to the substratum southwest of Baton Rouge, La., on the other hand, is as much as 100 feet, and the thickness of the substratum may be as much as 400 feet.

Although a shallow depth to firm substratum sands may be desirable from a foundation standpoint, the relatively high permeability of the substratum

---

1. Fisk, H. N., Geological Investigation of the Alluvial Valley of the Lower Mississippi River. Mississippi River Commission, Vicksburg, Miss., 1944.
2. Fisk, H. N., "Mississippi River Valley Geology Relation to River Regime." ASCE Trans. Vol. 117, 1952.

may often result in an expensive dewatering operation if deep excavations are involved. Pressure relief wells are often installed to minimize undesirable uplift pressures where excavations bottom in clays which overlie substratum sands at shallow depths. Where the excavation bottoms in the substratum, the problem of keeping the excavation free of water may be sizeable. Permeabilities of the substratum generally range from 400 to 2000 x 10^{-4} cm/sec, with the larger values associated with the deeper and coarser portion of the deposit.

Braided Stream Deposits

Although braided stream deposits cover large portions of the alluvial valley of the Mississippi (Fig. 1) they are usually so distant from the large streams that few major engineering projects have been concerned with them. Consequently, classification of various braided deposits and the soils associated with them has lagged behind that of the other major alluvial types found within the floodplain. Braided stream deposits were laid down by overloaded, shallow, many-channeled rivers during the earlier stages of valley alluviation. The sands and gravels of the substratum undoubtedly were deposited by such streams. As sea level neared its present stand, braided channels in the lower portion of the valley gradually changed into deep, single-channeled rivers. In the upper portion of the valley, streams retained their braided character and deposited appreciable thicknesses of fine-grained braided topstratum deposits.

Braided topstratum deposits normally consist of complexly interfingering lentils of materials ranging from sand to clay. Most common soil textures are sandy silt and clay sand. Depth to clean substratum sand and gravel ranges from 40 feet in the lower portions of the valley to only a few feet below the surface in the extreme northern part of the valley (see Fig. 2). These deposits are generally the oldest of the topstratum deposits; they are relatively dense and low in water content. Organic content is also low and, except under unusual circumstances, braided stream deposits make good foundations for engineering structures.

Meander Belt Deposits

Meander belt deposits are among the most important in the alluvial valley. They are disposed up- and down-valley in elongate bands that average from 10 to 15 miles in width. Because they flank the major streams where the most intensive engineering-soils explorations have been carried out, a considerable amount of data have been collected, which has permitted a rather comprehensive breakdown of depositional environments within the meander belts. Four major types are distinguished: (a) natural levees, (b) point bar deposits, (c) abandoned channel deposits of "clay plugs," and (d) abandoned course deposits.

Natural Levees

Natural levees, as the name implies, are natural dikes built on either side of a stream by the deposition of coarse materials contained in overflow waters. These levees rise to elevations of 15 or more feet above the

surrounding floodplain level at the water's edge and slope gradually away toward the lowlands on the landward sides of the levees. Predominant textures of natural levee deposits are sandy silt and silty clay. There is a decrease in grain size away from the crest of the levee and in a downstream direction; fine sand and silt commonly occurring in the northern part of the valley, clay in the southern portions. Because of their elevated position the soils are well drained. Water contents are usually low. Organic content is moderate to slight. They are normally too thin to be of consequence in the design of heavy structures but provide excellent subgrades for highways and airfields or borrow material for fills.

Point Bar Deposits

Point bar or accretion deposits are formed on the insides of river bends; actually whenever the channel migrates. Sandy bar deposits or "ridges" are laid down during high stages of the river and, as the water subsides, an arcuate depression or "swale" often flanks this bar on the landward side. The swale is modified by subsequent high stage flow and eventually is isolated as the river continues to migrate. The depression is gradually filled with fine-grained material and an alternating series of sandy ridges and clayey swales results, conforming to the curvature of the migrating channel. The ridges of point bar areas are characterized by sand at shallow depths. Sand within the ridges may reach the surface in the northern part of the valley. In the southern portions, depths to sand range from 10 to 30 feet. The clayey swales of the point bar or accretion areas sometimes reach depths exceeding 60 feet but more often are on the order of 30 feet. The lower portions of swales consist almost entirely of clay. Water contents are high and organic content moderate to high. The thin layer of soil covering both ridges and swales usually consists of relatively impermeable silty sands, clay silts and silty clays. Other soil types make up less than one-quarter of these deposits. Organic materials are confined to infrequent lenticular pockets.

Since shallow sands are often desirable for engineering structure foundations, point bar deposits are ideally suited in such instances. Subsurface conditions change within short distances laterally in these deposits, however, and the clayey swales must be delineated with great care in selecting sites for structures. The spacing of alternately impermeable swales and permeable ridges beneath and adjacent to massive artificial levees along the Mississippi River results in occasionally severe underseepage conditions which can be reliably predicted once the disposition of the alluvial deposits is known.

Abandoned Channel Deposits

Abandoned channels are arcuate segments of the river left as oxbow lakes on the floodplain when the river shortens its course. These lakes are gradually filled with fine-grained deposits. The segment abandoned may consist of an entire meander loop when the river cuts directly across the narrow neck between two converging arms of the loop, or it may be a portion of a loop cut off when the river chooses a larger swale during flood and abandons the outer portion. The former is known as a "neck" cutoff, the latter as a "chute" cutoff. Deposits filling abandoned channels are predominantly clay and silty clay. Shortly after cutoff, a wedge of sand fills

both arms of the abandoned channel at the point of cutoff and soon thereafter an oxbow lake is formed. The only material deposited within the oxbow lake is from periodic overbank flow from the river and as the channel migrates away from the point of cutoff only the finest materials find their way into the oxbow lake. Consequently, clays may fill these abandoned channels to the depth of the former active channel; clay 100 to 150 feet thick is not uncommon. Clays settling within deep oxbow lakes are characteristically high in water content and, since plants grow within the lake area only when it is filled or nearly filled with sediment, organic content is low except within the upper 10 feet of the clay body. Clay plugs or abandoned channel deposits are among the deepest clay bodies found in the alluvial valley. They are relatively unconsolidated and tend to exhibit high compressibility and low strength. Special design and construction measures are usually necessary where levees cross abandoned channel deposits. Their distribution often vitally affects the location of engineering structures and drainage and navigation channels. In addition they have a significant effect on river migration. Wherever these deep clay bodies occur along the river they act as "hard" points which radically alter the normal rate and direction of stream migration and greatly influence revetment placement and levee location.

Abandoned Course Deposits

Abandoned courses are lengthy segments of the river left when the stream abandons a meander belt and chooses a new course within the floodplain. They mark the position of the last channel the river occupied in the abandoned meander belt. The abandoned course, sometimes hundreds of miles in length, gradually fills with sediment and is often occupied by a smaller stream. An abandoned meander belt with a centrally located abandoned course of the Mississippi River is shown in Figure 2. Little is known about the details of the filling of abandoned courses. Indications are that the old course fills with a wedge of sand, thickest where the new course diverges from the old, and gradually thinning downstream. Subsequent stages involve the occupation of the abandoned course by smaller streams and its gradual filling with fine-grained deposits. The fine-grained deposits within the abandoned course are normally clays or silty clays. Organic content is relatively low. Soil densities vary directly with the age of the meander belt within which the course is located but are normally low. Physical characteristics of the abandoned course deposits are much the same as those of the abandoned channel. The most important difference is the normally shallower depth to sand within the former.

Backswamp Deposits

Backswamp deposits consist of sediment laid down in shallow ponded areas during overbank flow. The water may be trapped between high meander belt ridges or between a meander belt ridge and the valley wall. The coarser material from overbank flow is dropped near the stream to form natural levees. The finer material settles slowly in low-lying areas as the ponded water gradually drains off, seeps into the ground, or evaporates. Grasses, shrubs, and trees grow profusely in these areas and add to the organic content of the deposits. After each inundation of an individual

area of backswamp deposition, the clays may be desiccated so that even though backswamp deposits are generally finer than the clays of the abandoned channel they are denser and water contents of the former are notably lower. Backswamp deposits are some of the most homogeneous deposits found within the floodplain. More than 80 per cent of the deposits normally consist of clay. Thickness of the deposits increases in a down-valley direction and, where not underlain by meander belt deposits, the contact of the backswamp clays with the underlying sands and gravels is remarkably constant within any one area. Their normally high organic content make them occasionally undesirable as foundation material. Heavy engineering structures must often resort to pile foundations in underlying sands when located within areas of backswamp deposition. However, because they have been desiccated as a result of alternate wetting and drying, backswamp clays tend to be preconsolidated and settlements of structures resting directly on them are somewhat less than they would be if founded on abandoned channel deposits.

Deltaic Plain Deposits

The deltaic deposits are the result of the outbuilding of the land surfaces seaward by the various past deltas and the present delta of the Mississippi River. As soon as a delta of the Mississippi is abandoned, the sea begins to work its way inland. This process is aided by subsidence of the whole of the deltaic plain, both tectonically and as a result of gradual consolidation of the soft deltaic plain sediments. Yet the net result of the struggle between the advancing deltas on the one hand and the encroaching sea on the other is an over-all increase in the size of the deltaic plain. Rapidly increasing development of the area during the past decade has resulted in intensified study of the deposits and the engineering problems associated with them. Highways and causeways, locks and waterways, are part of an extensive transportation network. Levees and floodways have been built to protect the area from flooding.

In order to introduce properly the various environments characteristic of the deltaic plain, it is necessary to digress momentarily and describe the much older and firmer materials which underlie them—often at shallow depths. These are the Pleistocene deposits, ancient materials which flank and rise above the floodplain as terraces in the northern part of the valley but gradually slope beneath the surface of the floodplain farther south. Beneath New Orleans these deposits lie at depths as shallow as 50 feet and, as shown in Figure 3, slope gradually northward and emerge at the surface north of Lake Pontchartrain.

To properly visualize this surface, it must be imagined as a former floodplain of the Mississippi River before the advent of the last glaciers. As the glaciers grew, sea level dropped and the surface was subjected to erosion, desiccation, oxidation, and consolidation for tens of thousands of years. As the ice began to recede, sea level rose and the former floodplain became covered by the sea. It is on this relatively firm surface that the considerably younger and less consolidated deltaic plain has been built. A shallow depth to Pleistocene deposits is often the deciding factor in site selection for heavy structures. Densities of the Pleistocene deposits are high, water contents low, organic deposits compacted and fairly scarce. Soil textures range from clay to sand and gravel. Shear strengths of Pleistocene materials are normally 1000 lb per sq foot or greater.

Overlying the Pleistocene deposits are the sediments of the deltaic plain. During the past several years attempts have been made to arrive at a comprehensive classification of the important environments of deposition within the deltaic plain. Of particular importance was the development of a scheme of classification that had significance from the standpoint of engineering soils. The classification listed below is based on a number of sources. It is tentative and probably will be revised as studies continue.

Deltaic Environments of Deposition

- Lacustrine environment
- Bay-sound environment
- Reef environment
- Delta environments:
  - Pro-delta
  - Distributary bar fingers
  - Interdistributary troughs
- Marsh environment
- Swamp environment
- Beach environments:
  - Sand beaches
  - Shell beaches
- Stream environments:
  - Point bar
  - Natural levee
  - Abandoned course
  - Abandoned distributary
  - Tidal channel

A comprehensive discussion of each of these environments and what is known of the types of deposits resulting from each is beyond the scope of this paper. Among the more important are those resulting in deposition of sands. These are found mainly in the sand beaches. Sand beaches 15 or more feet high are found today, particularly in areas just offshore of the ancient sand-depositing deltaic systems. The Chandeleur Islands and Grand Island series of beaches northeast and west of the present delta are cases in point. Even more important from an engineering standpoint are ancient sand beaches now partially or completely buried by other deltaic deposits. Partially buried sand beaches are called cheniers, and are common along the coast of southwest Louisiana. Completely buried beaches are known to exist in a number of places within the deltaic plain. One is shown in Figure 3 directly beneath the city of New Orleans where it affords a relatively good foundation at fairly shallow depths.

Only slightly less important from the standpoint of foundations are the shell beaches and the shelly reef environments. Shell beaches consist largely of shell fragments intermixed with from 25 to 50 per cent sand. Like the sand beaches they form relatively small, lenticular, discontinuous bodies completely or partially buried beneath less stable deltaic plain deposits. Reefs, on the other hand, may be extensive, sometimes covering many square miles and reaching thicknesses of 10 ft or more. The shells in the reef environment are usually entire rather than fragmented as in the case of the shell beaches, but may occur together with considerable amounts of clay and thus are normally less desirable as foundation material.

The swamp and marsh environments of deposition generally are comprised of fine-grained more or less organic soils. Swamps are those areas characterized by growing trees. Marshes contain only occasional trees and extensive amounts of sedges and grasses. Organic content is highest within

the marsh environment. Layers of peat and humus up to 25 feet thick are known to exist within some of these environments. Organic soils associated with swamp and marsh environments are among the most treacherous within the deltaic plain. In almost every instance, organic materials must be removed prior to even light-weight construction. A costly phase in the replacement of U. S. Highway 51, which follows the narrow neck of land between Lake Maurepas and Lake Pontchartrain (see Fig. 3), has been the removal of an almost uninterrupted 10-foot layer of organic material. The old highway was built on fill placed directly on the swamp deposits. The many small bridges crossing drainage channels along the highway were placed on piles sunk to the shallow Pleistocene deposits. Over the years the roadway has subsided as much as 5 feet into the marsh; the bridges have remained at essentially the same level at which they were built. As a result, travel over the old highway was roughly comparable to a ride on a roller coaster.

The gradual decay of organic materials beneath the deltaic plain results in the production of considerable quantities of marsh gases. The accumulation of gases in subsurface deposits often augments the water pressures that exist therein. These pressures sometimes can cause blow-ups in foundation excavations if adequate precautions are not taken. It has been noted in some cases that gases bleed up through open-grained timber piling penetrating gas-bearing strata. This can cause damage to freshly poured concrete around the pile caps unless preventive measures are taken. Gas has also been noted bleeding through construction joints in the base of lock structures in the southern Louisiana area. Some large buildings in New Orleans are provided with a sand filter blanket to bleed off excess marsh gases and thereby reduce uplift pressures.

A final type of deposit that should be considered in even a cursory discussion of deltaic plain deposits is the delta environment, the prime builder of the land surfaces. Nearly all of the inorganic deposits which make up the deltaic plain resulted from original deposition in an active delta or subsequent reworking of these deposits by the sea. Five major deltas in addition to the present delta have been active in building the deltaic plain. These abandoned deltaic complexes swing in a 200-mile arc across the entire southern border of the plain. Recent work in the present Mississippi delta has gone far toward classifying the environments found there. Preceding each deltaic advance is a sequence of pro-delta clays and silty clays which gradually become coarser upward in the section. Above these are distributary bar fingers, wedges of fine sand which build seaward, finger-like, as the distributary advances. Between these fingers of sand are pockets of fine clays and silts, the interdistributary trough deposits. Intensified activity by oil interests in the shallow offshore Gulf areas has resulted in comprehensive studies of the engineering characteristics of these sediments. Heavy, expensive drill rigs must be firmly anchored to the Gulf bottom before successful drilling can begin. Changes in soil strength due to varying clay mineral types, the changes in strength brought about by flocculation of certain clay particles on contact with a saline environment, the effect of disintegrating organic material on the strength of silts and clays, and the correlation of sediment types with the environment in which they are deposited, are all problems the solutions of which will have direct and valued applications to the design of structures in shallow offshore areas.

Selected Engineering Applications

As can be seen from the preceding discussion, a knowledge of the various environments of deposition within the alluvial valley is a necessary prerequisite for the prediction of soil types and, in turn, their effects on foundation conditions. Table 1 lists a group of these environmental types, the soil types normally associated with each, and representative ranges of physical properties of engineering significance. The significance of the various geologic deposits in the lower Mississippi valley with respect to engineering foundation problems has been mentioned briefly in appropriate places earlier in this paper. Of equal importance are a number of related engineering problems which also are dependent on the geology of the lower Mississippi valley. Among these can be mentioned: (a) the location of aggregate sources, (b) the prediction of river stability, and (c) the prediction of underseepage or dewatering conditions.

Aggregate Sources

Aggregate sources are a particularly troublesome problem throughout the alluvial valley. Toward the north and near the valley walls a knowledge of the sequence of terrace alluviation is a prime requisite to locating the extensive sand and gravel deposits that occur there. Point bar deposits, particularly the active point bar deposits along the river, are the principal source of sand and gravel within the floodplain.

Aggregates are at a premium farther south. South of the latitude of Baton Rouge, river-deposited sands are too fine for use as aggregate and gravels are not to be found. Here another source, the shell beds of the shell beach and reef environments, assumes considerable importance. A knowledge of the disposition of the various former deltas of the Mississippi allows the elimination of search within large areas where such buried shell reefs are improbable. Careful study of limited borings and a knowledge of the growth characteristics of modern reefs permit fairly accurate guesses as to the extent of buried shell beds.

Of increasing importance within the southern part of the valley are lightweight aggregates produced by burning clays at high temperatures. Certain types of backswamp clays appear to be most desirable for use in such aggregates. Their grain size and clay mineral content, coupled with the proper amount of organic matter, make them ideally suited for use in what appears to be a fast-growing industry. The organic matter becomes gaseous during the burning process and expands the material to a light durable brick-like aggregate that can be used satisfactorily in a variety of concrete structures.

River Migration and Channel Diversion

Of considerable importance to engineers entrusted with site location along the banks of the Mississippi River is a valid prediction of the direction the river may migrate during the life of an installation. Factories, anxious to utilize cheap transportation facilities provided by the river, bridges, pipe lines, overbank flow structures, docks, and levees, are examples of the numerous installations that are affected by the stability of the river. Based on the mapped record of river migration during the past 200 years, together with evidence from borings and airphotos, geologists have been able to

delineate with remarkable success the position of the river during the past 3000 years. Refinements of these studies enable predictions of future river migration based on a knowledge of rates of river meandering as controlled by the sediments along the river bed and banks that are equally rewarding where such methods have been tried. An example of an important application of such geologic studies has been recent work on the imminent diversion of the Mississippi River through its Atchafalaya distributary. To the geologist this impending diversion, with its serious consequences, is merely the latest in a series of similar occurrences in the normal sequence of floodplain formation. Geologists working with engineers were able to establish the probability of such a diversion and to predict the time when such a diversion would become critical. Several large engineering works are now being planned and constructed by the U. S. Army Corps of Engineers to prevent such an occurrence.

Underseepage

Seepage and sand boils landward of Mississippi River levees have been a problem during major high waters. One of the major contributory causes of this condition is the presence of the pervious sand substratum which is cut into by the river, thereby permitting nearly the full river head to be transmitted in the form of hydrostatic pressures acting on the base of the relatively impervious topstratum. These high pressures have been found to exist for considerable distances away from the river. Where the topstratum is thin, as in point bar ridge deposits, the potential danger from underseepage is increased. There is usually little or no danger from underseepage where thick clays, such as backswamp and abandoned channel deposits, are found. The configuration of point bar swales and abandoned channels with respect to the levees also has an important bearing on underseepage. The greatest concentration of seepage occurs along the riverward edges of swales and channels, and in instances where the long dimension of these features is parallel to or intersects the levee at a small angle and seepage is concentrated between them and the levee toe. Measures which have been or are currently being used to protect levees against excessive underseepage include sublevees, berms, and relief wells.

TABLE 1

TYPICAL PROPERTIES OF SELECTED ENVIRONMENTS OF DEPOSITION WITHIN THE MISSISSIPPI ALLUVIAL VALLEY

ENVIRONMENT	GRAIN SIZE & ORGANIC CONTENT (1)	PREDOMINANT SOIL TEXTURE (2)	NATURAL WATER CONTENT, PER CENT	LIQUID LIMIT	PLASTICITY INDEX	SHEAR STRENGTH (3)	
						COHESION LB PER SQ FT	ANGLE OF INTERNAL FRICTION, DEG
BRAIDED STREAM (4)		CLAY SANDS (SC) TO SILTY CLAYS (CL) SANDS (SP)	25-40 —	30-75 NP	10-55 NP	200-1100 0	30 30-40
NATURAL LEVEE (5)		CLAYS (CL) SILTS (ML)	25-35 15-35	35-45 NP-35	15-25 NP-5	360-1200 180-700	0 10-35
POINT BAR (RIDGES) (6)		SILTS (ML) AND SILTY SANDS (SM)	25-45	30-55	10-25	0-850	25-35
ABANDONED CHANNEL (6)		CLAYS (CL & CH)	30-95	30-100	10-65	300-1200	0
BACKSWAMP (7)		CLAYS (CH)	25-70	40-115	25-100	400-2500	0
SWAMP (8)		ORGANIC CLAY (OH)	110-265	135-200	100-165	VERY LOW	
MARSH (8)		PEAT (PT)	160-465	250-500	150-400	VERY LOW	
PRO-DELTA (8)		CLAY (CL & CH)	20-120	25-95	10-80	175-300	0
LACUSTRINE (8)		CLAY (CH)	45-165	85-115	65-95	75-150	0
BEACH (8)		SAND (SP)	SATURATED	NP	NP	0	30
BAY-SOUND (8)		CLAY SILTS AND SILTY CLAYS (CL & CH)	20-70	40-80	25-65	250-700	15-20
SUBSTRATUM		SAND (SP)	SATURATED	NP	NP	0	30-38
PLEISTOCENE (8)		CLAYS AND SILTY CLAYS (CL & CH)	15-50	25-80	20-75	550-5000	0

(1) LEGEND: GRAVEL (>2.0 mm); SAND (2.0–0.06 mm); SILT (0.002 TO 0.06 mm); CLAY (<0.002 mm); ORGANIC MATERIAL.

CHARACTERISTIC GRAIN SIZES FOR BRAIDED STREAM, NATURAL LEVEE, POINT BAR, BACKSWAMP, AND ABANDONED CHANNEL DEPOSITS BASED ON FISK, H. N., FINE-GRAINED ALLUVIAL DEPOSITS AND THEIR EFFECTS ON MISSISSIPPI RIVER ACTIVITY. MISSISSIPPI RIVER COMMISSION, VICKSBURG, MISS., 1947.

(2) SYMBOLS BASED ON UNIFIED SOIL CLASSIFICATION SYSTEM.

(3) SHEARING STRENGTHS OF CLAYS BASED ON UNCONFINED COMPRESSION TESTS.

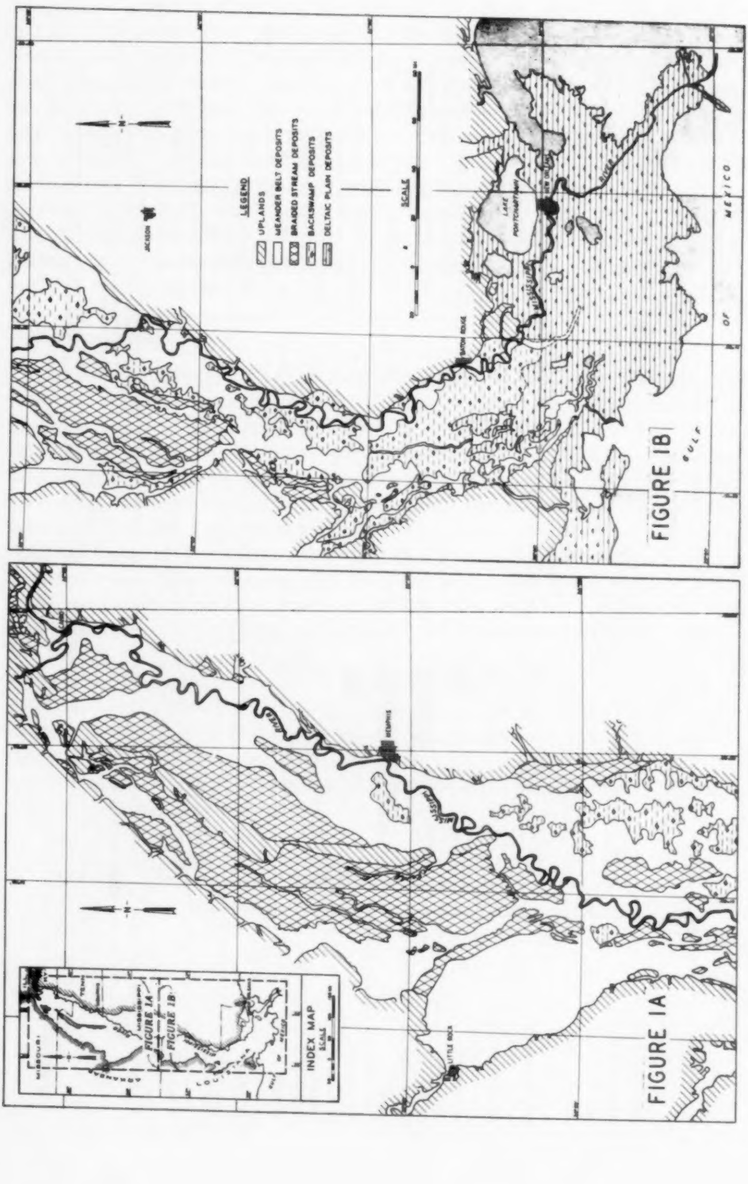
(4) DATA OTHER THAN GRAIN SIZE BASED ON 3 BORINGS AT HAYNES AND DEVALL BLUFF, ARK.

(5) DATA OTHER THAN GRAIN SIZE BASED ON APPROXIMATELY 25 BORINGS IN SOUTHERN PORTION OF VALLEY.

(6) DATA OTHER THAN GRAIN SIZE BASED ON APPROXIMATELY 100 BORINGS IN VALLEY SOUTH OF MEMPHIS, TENN.

(7) DATA OTHER THAN GRAIN SIZE BASED ON APPROXIMATELY 150 BORINGS SOUTH OF NATCHEZ, MISS.

(8) DATA OTHER THAN GRAIN SIZE BASED ON APPROXIMATELY 200 BORINGS IN NEW ORLEANS AREA.



DISTRIBUTION OF DEPOSITIONAL TYPES IN THE LOWER MISSISSIPPI VALLEY

Fig. 1

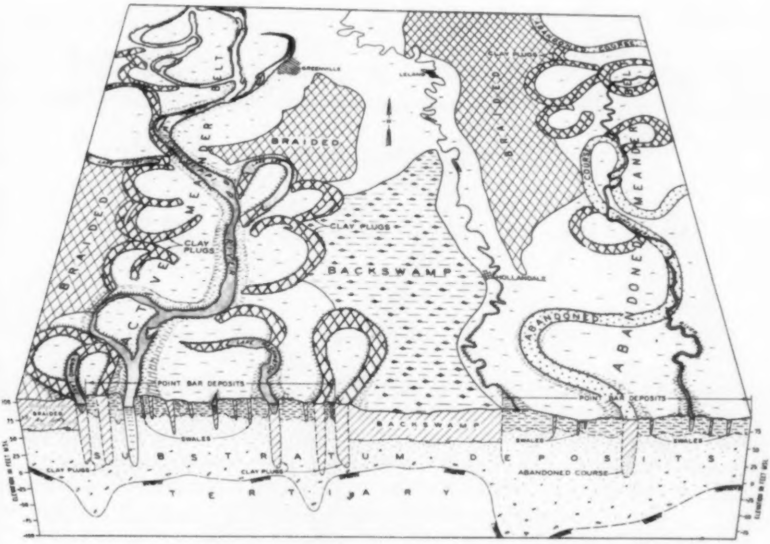


Fig. 2. Major environments of deposition and associated soil types in the vicinity of Greenville, Mississippi. On the subsurface profile dots represent sand, slant lines clay, and dashed lines silt.

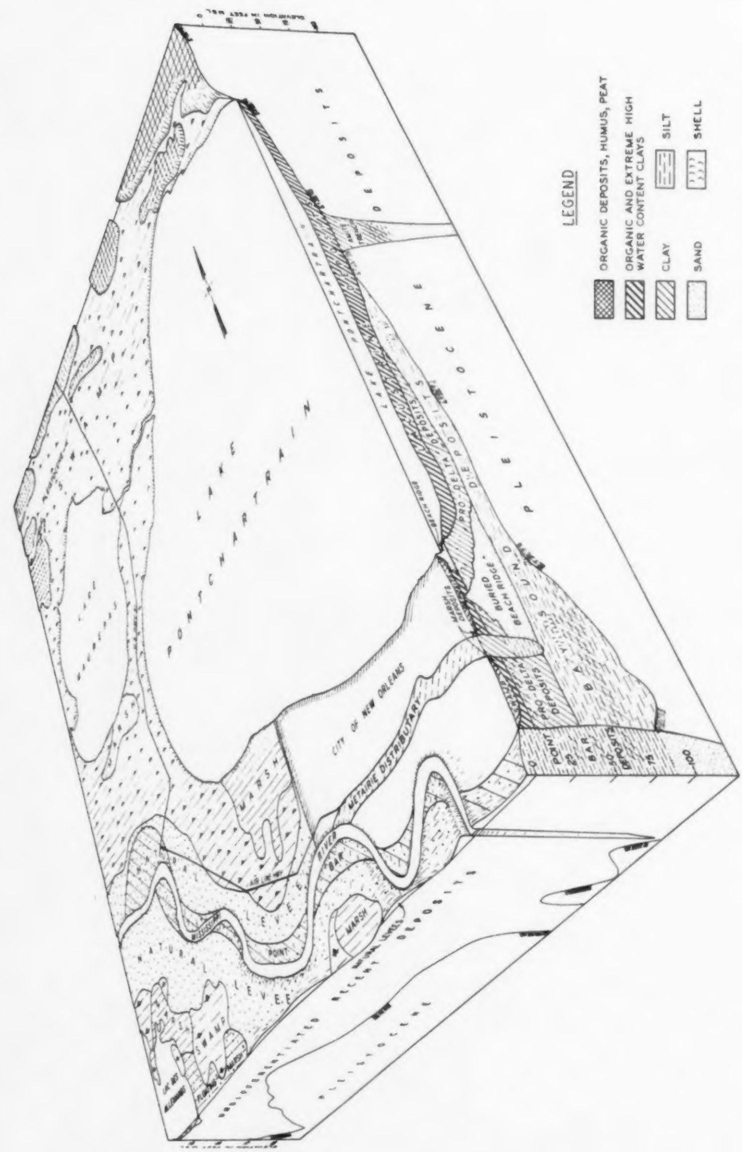


Fig. 3. Major environments of deposition and associated soil types in the vicinity of New Orleans, La.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

DEWATERING MIAMI'S BISCAYNE AQUIFER

Byron J. Prugh,[1] A.M. ASCE
(Proc. Paper 1299)

SYNOPSIS

Foundation excavations in the extremely permeable Biscayne Aquifer underlying Miami previously have usually been constructed by braced sheet piling with a tremie seal. Wellpoints in the past have been successfully used to dewater the top sands and oolitic limestone. This paper describes the use of wellpoints in dewatering a major excavation penetrating deep into the Biscayne Aquifer through the Miami oolite and the soils encountered.

INTRODUCTION

The Biscayne Aquifer underlying Miami and surrounding parts of Dade County, Florida has been referred to as one of the most highly permeable aquifers ever investigated by the U. S. Geological Survey and is comparable with coarse, clean, well sorted gravel. Typical overall permeabilities range from 10,000 to 32,000 x 10^{-4} cm/sec for the 40 to 200 feet thickness of this strata. An average 6" well with a few feet of open hole below the end of the casing will yield more than 1,000 GPM, if the bottom of the casing is from 40 to 120 feet below ground surface.

No large open excavation had ever been unwatered in this area without some form of tremie concrete bottom being utilized or enormous volumes of water being pumped. It had been considered doubtful that unwatering in this strata could be successfully and economically accomplished. Properly installed and operated wellpoint systems had been suggested in the Biscayne Aquifer to stabilize the sand layers, producing firm side slopes and workable subgrades.[2] Loss of fines and disturbance of the natural soil would also be reduced to a minimum.

Note: Discussion open until December 1, 1957. Paper 1299 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83, No. SM 3, July, 1957.

1. Asst. Chf. Engr., Moretrench Corp., Rockaway, N. J. and American Dewatering Corp., New York, N. Y.
2. "Unusual Foundation Conditions in the Everglades," by Paul H. Shea, Trans. ASCE, Vol. 120, 1955, p. 101.

The Northwest Fourth Street Sewage Pump Station for the City of Miami's sewage disposal system (see Figure 1) was roughly 61' x 115', with the main area's lowest subgrade at Elevation 75.11 or 25.15 feet below Mean Sea Level. (El. 100.263 Miami City Datum). The pump station is located alongside the retaining wall of the Miami River where NW 4th Street intersects NW North River Drive. Tidal fluctuations during the dewatering of the foundation ranged from Elevation 99.9 to Elevation 103.2. This placed subgrade of the structure well into the Biscayne Aquifer with an average head of about 27 feet of water.

Biscayne Aquifer

Approximately 100 to 200 feet thick, it is made up of various strata laid down in the Cenozoic era, mostly during the Quaternary period, Pliestocene epoch. (See Figure 2). It rests on the so-called 400 foot thick Floridan aquiclude, rocks of the Miocene and early Pliocene epochs, that provides an almost watertight foundation. These rocks, while slightly permeable, do not furnish sufficient supplies for normal wells and act as a confining cap or relatively impervious layer. Under this cap is the Floridan aquifer, composed of Hawthorn, Tampa, Suwannee and Ocala limestones of the Miocene, Oligocene and Eocene epochs. The water in the Floridan aquifer is artesian with a piezometric head which will cause water, in tightly cased wells, in the Miami area to rise to about 40 feet above Mean Sea Level. The water, however, is considered too saline and corrosive for most purposes.

The main components of the Biscayne Aquifer from top to bottom are:

(1) "Pamlico Sand" - a thin veneer of fine black to white sand which was deposited during the last glacial period (Wisconsin). In the Miami area, depths of 20 to 50 feet of this sand have been found on other wellpoint jobs in solution holes or old tidal channels cut in the underlying oolite and especially in the dunes along the coastal ridge. In general, the sand layer is only a few feet thick and at this job site was one foot thick with solution holes going an extra one to two feet. (See Figure 3) No appreciable quantity of water is supplied by this layer.

(2) "Miami Oolite" - The typical Miami oolite is a soft white oolitic limestone containing as much as 95% of calcium carbonate. Locally it has layers of calcite and more or less quartz sand (mostly more at this job site). It's oolitic name is derived from the large proportion of spherules or small round grains of calcium carbonate deposited concentrically around a quartz grain nucleus. The oolitic limestones underlying Miami have a much lower proportion of oolites than the same strata of limestone dewatered further south at Key West.[3]

The top layers were probably laid down during Talbot time in shallow water on top of a bar. This bar now forms part of the coastal ridge and underlies all of Miami. Depositation on this sloping bar produced laminated cross-bedding of layers. Percolating waters have in many places dissolved the oolites and have riddled the rock with solution holes and channels. Fluctuating water levels have caused horizontal layers of redeposited material. The

3. "Wellspoints Master Oolite," by E. Day Wood, Jr., Engineering News Record, May 6, 1954, p. 35.

numerous solution holes allows a high rate of vertical recharge from rainwater. Macrophotographs (Figure 4A) plainly show the white spherical oolites interbedded with subrounded quartz grains.

(3) "Fort Thompson Formation" - This formation is composed of layers of marine and freshwater limestones and some quartz sand and shells. At the job site there were extensive layers of nearly pure white and brown quartz sands with a few limestone fragments. These alternated between layers of hard porous grey to white limestone. Tests run on this calcareous sandstone (or sandy limestone) found near subgrade showed the horizontal and vertical permeabilities were nearly equal. One sample did give a ratio of 7 horizontal to 1 vertical but this was an exception. Over a distance of one foot in a typical sample, the composition could vary from nearly pure calcite crystals and subsequent dense structure to a highly pervious latticework or honeycombed structure with a sand content as high as 36%. Previously this formation was classified as the Tamiami formation, but recent geological interpretations of additional samples taken from borings and wells have caused it to be identified as part of the Fort Thompson formation due to the occurrance of freshwater limestones, although the sand content is unusually high.[4] In fact, seams of pure sand were encountered at the job site.

A macrophotograph (Figure 4B) made by focusing on top of a level sample shows the cemented subangular quartz grains (black) and large void areas (white) to give the highly permeable honeycombed structure.

(4) "Tamiami Formation" (Top Part) - The Tamiami formation is a marine limestone that has a very high sand content and which at times approaches a nearly pure quartz sand. Color is greyish white to tan. Numerous marine shells aid in distinguishing it from the Miami oolite and Fort Thompson formation. This formation was not penetrated by the wellpoints at this job site.

The above formations make up the Biscayne Aquifer. The aquifer is in effect a large wedge that had the thick side along Biscayne Bay and thins westward to the Big Cypress Swamp.

The preceding is a general description of the geological formations that underlie Miami including the NW 4th Street area and were responsible for the extremely large flow of water encountered. It should be remembered that within the two major water bearing stratas, the Miami oolite and Fort Thompson formation, the composition and properties were found to be extremely variable in both vertical and horizontal directions. Others have pointed out this fluctuation over very short distances and as also was noted in the series of tested samples above. Over 100 rotary drill borings were made at the job site during the installation of grout pipes. Many of them were not more than five feet apart, but all varied considerably, especially in the hardness, thickness and location of the rock layers.

While an expensive procedure, continuous core borings are needed to give an accurate picture of the sub-surface conditions. The closer these borings are together, the better. A maximum distance of 30 feet between borings is suggested to obtain accurate information to interpret for design and construction procedures in a variable soil such as encountered on this job site.

4. "Age of Subsurface Tamiami Formation Near Miami, Florida" by Nevin D. Hoy and Melvin C. Schroedar, The Journal of Geology, Vol. 60, No. 3, May 1952.

As stated in a previous article[5] any bore hole within the structure or excavation lines should be grouted when completed. This is especially true when penetrating into a pressure stratum or into an aquifer that will become a pressure area when dewatering for construction purposes will create differential hydraulic pressures. In this case an ideal one exists, the Fort Thompson formation. The water in the more pervious layers in the Fort Thompson is not under pressure, but the open bore holes removes the friction of the less pervious material above and allows flow vertically upward without loss in head. The grouting is for the purpose of filling up only the boring hole so no opening exists directly from the aquifer to the excavation construction areas above. In deep lying aquifers, the cost of pressure relief or grouting is excessive after excavating has revealed the presence of an open bore hole.

If a correct interpretation was made, the four preliminary borings at NW 4th street, taken near the four corners of the structure, were indicative of the soil at that location. In the 60 to 120 feet between borings, considerable variation of the soil was found which necessitated on the job changes in the dewatering procedure and materially increased dewatering installation costs.

Sheeting

The pump station not only penetrated the Biscayne aquifer but was built in a restricted area between NW North River Drive and the Miami River. To allow adequate working space and to protect the river side, the job was sheeted as shown in Figure 1. Diagonal "H" beams were used as bracing from the top waler at El. 102 on the river side to the first full set of bracing struts at El. 94.5 (See Figure 5). Until excavation for the first full set of walers and braces was completed and these diagonals installed, temporary diagonals to support both the top and middle walers were butted against short lengths of steel sheet piling driven into the limestone in the center of the excavation. (See Figure 6A and 6B) Triangular brackets (See Figure 6B) placed on top of the riverside walers and welded to the steel sheet piling, prevented the waler from moving upward due to the vertical force component of the diagonal brace.

Dewatering Setup

More trench wellpoints were connected to two rings of header pipe. (See Figures 1 and 5) The top header at El. 90 to 92 was 16" in diameter with 2" and 4" inlets and surrounded the entire area including the "L" shaped working area on the North and East sides from which the cranes worked. As this header pipe was above all subgrades with the exception of part of the office floor slab at Elevation 94.54, it was permissable to open pump part of the excavation to allow this upper header to be installed below ground water level normally at Elevation 102.

Two sumps in the SE and NW corners handled the top water coming in horizontally above the upper header pipe through the laminated Talbot layers of Miami oolite. The proximity of the Miami River undoubtedly added to this

5. "Anchorage Excavation Test Versatility of Wellpoints," by Byron J. Prugh, Civil Engineering, September 1954, p. 48.

flow as considerable increases in the flow from these sumps were noted during high tide periods and with no appreciable time lag.

This top header allowed the placing of the lower 16" and 12" diameter header at El. 82 to 83 below the third set of walers at El. 86.9. An unusually heavy lateral flow was encountered on the North side coming through the limestone between the two headers. A trench, 3 feet wide and 8 feet deep, was dug along the center of the North side working area and a short distance down the east side working area, and was filled with a sand-bentonite mixture which partially reduced the lateral flow. One 75 foot header spur line ran north off the center of the south side lower header. The wellpoints on this spur were placed in the wall between the two wet wells and were concreted in place, later being grouted solid.

A temporary sheeted construction sump was placed on the northwest side. Originally, this sump took care of the lateral flow not stopped by the bentonite trench. It was later used to temporarily handle the large boil until the boil was stopped by grouting and finally, when the top wellpoint stage was shut off, it handled water seepage until the wellpoint equipment was removed.

Vicissitudes

When the lower header pipe, including spur, was first placed in operation, the static water level lowered almost to subgrade around the perimeter but below subgrade in the center. (See Figure 5) This is the reverse of the normal interior drawdown curve for wellpoint dewatering. Pinboils were observed plus one large boil in the NW corner of the main area, which fortunately was outside of the building line. The excavation was immediately flooded to El. 95.0 to prevent any loss of fines or reduction in the bearing capacity due to lowering of the relative density by disturbance of the soil. A short row of steel sheet piling was placed to isolate the large boil outside of the structure. This boil incidentally, has a static head that extended well above the top header pipe.

During this period, investigations were continuing which revealed that extensive fracturing of the limestone had taken place during the driving of the steel sheet piling, especially on the river side. This was aggravated by the presence of several extremely porous rock strata with large voids. Several of the auger rotary borings taken between the riverside sheeting and retaining wall had not only no recovery but a conical depression around the drill hole was the final result due to the filling of the voids with fragments displaced by the auger. It became evident that there was a direct connection with the river which was confirmed when live eels and shrimp were observed in the flooded excavation.

After flooding of the excavation to normal ground level, a partial grouting treatment was applied in the form of a partial exterior curtain wall. Grout placed under controlled pressures through drilled grout pipes was composed of varying mixtures of cement, alfesil, sand and Intrusion Aid. This grouting effectively cut off approximately 75% of the flow of water coming in through the open and fractured rock at the higher levels above subgrade. Grouting in the uniform fine sand layers was ineffective due to the fineness of the grains, whose average D_{10} size was .16 mm (D_{50} was .24 mm.)

Grout pipes were installed from the surface of the ground and varied in depth from 50 to 70 feet, depending on the ground surface elevation or the rock structure.

Observation of the grout pattern on the river side was done by carefully plotting the locations and elevation of areas and strata where negligible grout pressure and grout with a large sand percentage was used. These observations revealed that the fractures or open channels feeding the large boil were located under the sheeting about 30 feet downstream from the actual boil location.

When pumping was resumed, the wellpoint equipment was then able to completely control the flow of water coming through the bottom of the excavation and the contractor was able to pour his concrete foundation on a satisfactory bottom. Fortunately, no hurricanes struck the area during construction but several extremely heavy rains occurred.

Equipment

Over 300 - 2 1/2" Chelsea type wellpoints were used on the job in connection with several 4" Chelseas in critical spots. Chelsea type wellpoints placed in the fine to medium sand layers produced from 100 to 265 GPM when correctly installed with a filter medium.

10" and 12" diesel powered wellpoint pumps were used throughout the job. A maximum volume of about 24,000 GPM was pumped before the partial grouting reduced it to the expected flow through the undisturbed soil. Wellpoints were installed by the use of a Moretrench Holepuncher, Rotary Drill Rigs and a #4 Jetwell pump which had a capacity of 500 GPM at 240 PSI.

Filter Material

A dual purpose filter material was needed for placing around the wellpoints, a filter that was (1) graded to meet the criterions for a filter for the natural uniform fine sand layers, yet (2) with a very high permeability to carry the large flows of the honeycombed and fractured limestone layers.

Various filter materials were field tested and tried, the final filter material used being developed by the pumping contractor and consisted of synthetic oolites produced as a by-product of a water softening plant. The oolites were of a very uniform grain size distribution. As shown in Figure 7, nearly all oolitic grains passed the #10 sieve but were retained on the #40 sieve. Permeability, in place, was approximately 3,000 x 10^{-4} cm/sec. Laboratory tests substantiated by field performance indicated that the use of oolites as a filter medium produced more satisfactory results than normal filter sands due to the large void ratio and permeability. Macrophotographs of these materials are shown in Figures 8A and 8B. Oolitic filter material allowed a large quantity of water to flow to the wellpoint through the filter medium with little friction loss. Synthetic oolites are now being used extensively by contractors for wellpoint installations in the Miami area.

While salt water intrusion from the Miami River produced a high corrosiveness in the ground water being pumped which required constant vigilance to avoid leaks in the wellpoint system, the synthetic oolitic filter was not effected. It is felt that pumping over an extended period of time would cause some dissolution, as like any calcium carbonate rock, the synthetic oolites are attacked by acidic waters.

Removal

The "backing out" process was carefully planned and executed to avoid excess hydrostatic pressures that would endanger the structure by uplift or lateral pressure on the cantilevered walls and also prevent boiling in the compacted backfill around the structure.

During the "backing out" procedure, part of the north lower header line was removed to allow the screening room slab at El. 84.0 to be poured, therefore a second spur line (See Figure 1) was run off the top header along the north side of the screening room as a substitute for the removed lower header.

All foundation slabs were poured on undisturbed soil except in one small section where six piles were driven in an area previously excavated and backfilled. The header pipes and wellpoints were only removed after backfill, hydraulically compacted, was placed up to the underside of the header pipe in each stage.

CONCLUSION

While this particular job did entail certain unusual conditions for the dewatering contractor, it is felt that the lessons learned in this Biscayne Aquifer will be of great value on future work in this area, viz;

a) Larger structures in a very pervious formation such as the Biscayne Aquifer can be economically dewatered with wellpoint systems, provided that correctly designed equipment and installation techniques are used.

b) Adequate advance soil information can materially reduce the cost of dewatering.

c) The method of applying partial curtain grouting in very porous formations and stratified material can be economically used as an adjunct to wellpoint dewatering. Correct patterns, drilling, mixes and techniques are necessary for economic feasibility.

d) The use of correct filter materials and proper placing is essential for dewatering large flows in porous rock and sand.

e) Inspection by engineers of the excavation bottoms "in the dry" and without danger of soil disturbance can be made to determine the suitability of the foundation subgrades.

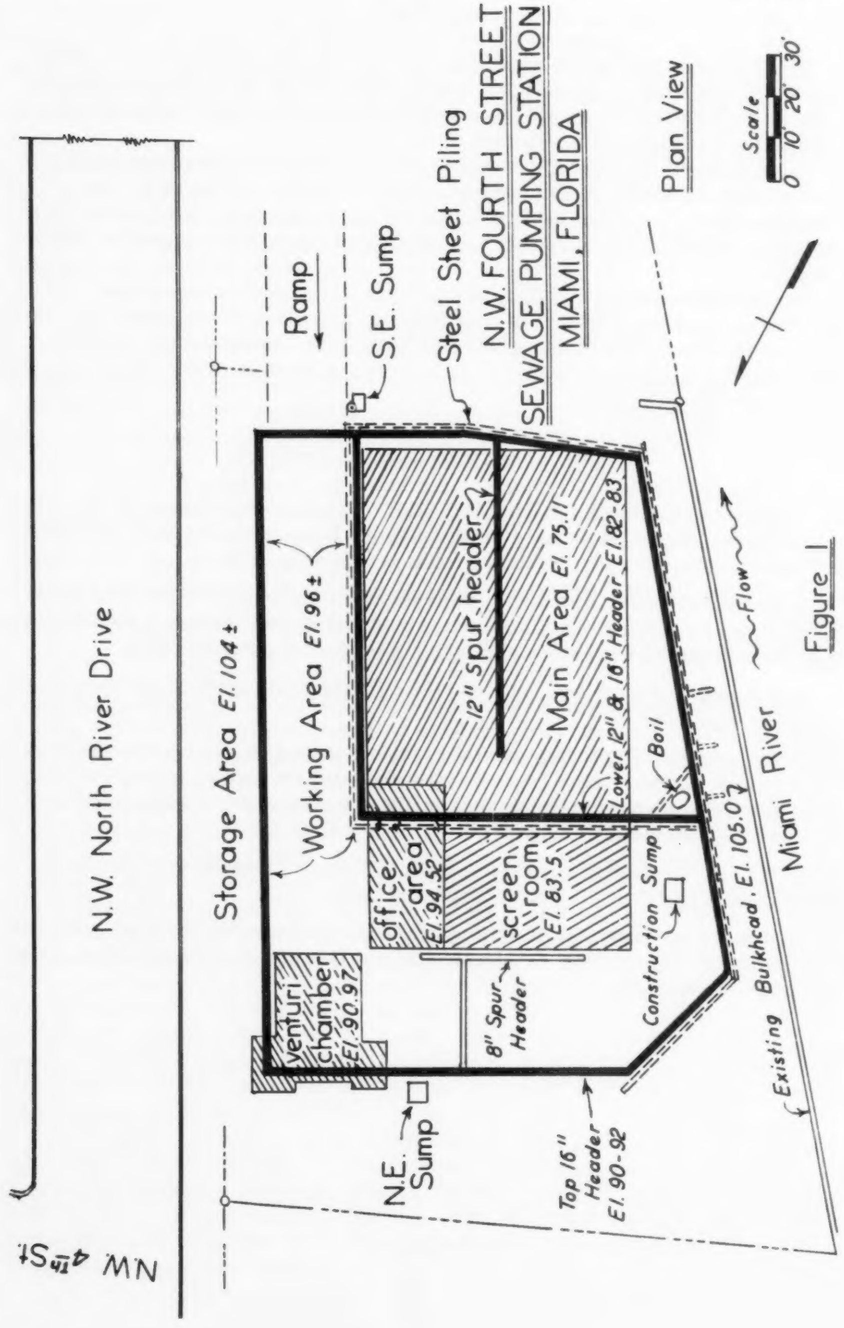

N.W. FOURTH STREET
SEWAGE PUMPING STATION
MIAMI, FLORIDA
Plan View
Scale
0 10' 20' 30'
N.W. North River Drive
N.W. 4th St.
Storage Area El. 104±
Working Area El. 96±
Ramp
S.E. Sump
Steel Sheet Piling
12" Spur header
Main Area El. 75.11
Lower 12" & 16" Header El. 82-83
Boil
office area El. 94.52
screen room El. 83.5
venturi chamber El. 90.97
8" Spur Header
Construction Sump
N.E. Sump
Top 16" Header El. 90-92
Existing Bulkhead El. 105.0
Flow
Miami River

Figure 1

Soils of the Cenezoic Era

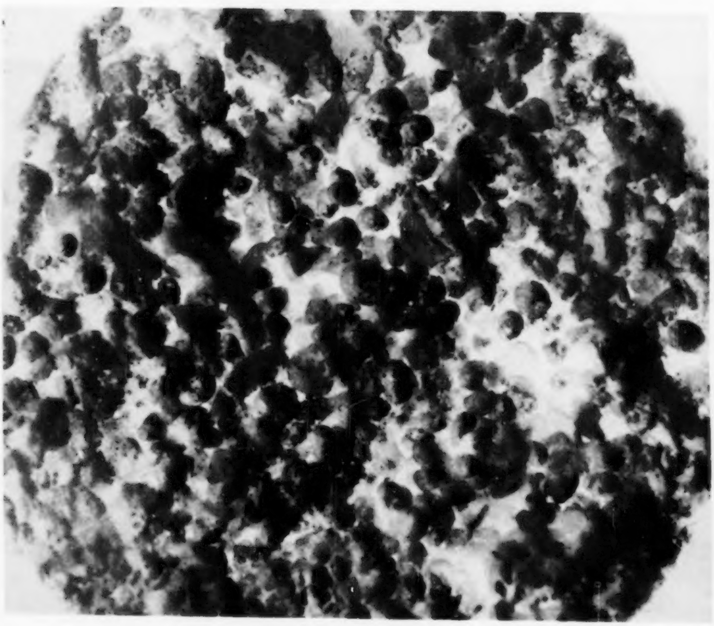
Schematic Geology Chart of the Miami, Florida Area at NW 4th Street Pump Station

Period	Duration in millions years	Epoch	Age	Time Shore Line	Depth water Formed in	Name	Soil	Thick-ness	Layer	Depth
		Recent	Present		Man Placed	FILL	Sand and rubble	3-5'		5'
Quatenary	2	Pliestocene	Wisconsin (G)		4' to 24'	PAMLICO SAND	Fine black sand	1-2'	Biscayne Aquifer	6'
			Sagamon	Talbot	42'	MIAMI OOLITE	Cross bedded in layers	25' to 30'		30'
				Penholoway	70'		Soft white sandy limestone and sand			
				Wicomico	100'					
			Illinoian (G)			FORT THOMPSON FORMATION	Marine and freshwater sandy limestones and sands	120'		150'
			Yarmouth	Sunderland	170'					
				Coharie	215'					
			Kansan (G)							
			Aftonian	Brandywine	270'					
			Nebraskan (G)							
Tertiary	58			Deep enough to prevent violent wave agitation		CALOOSAHATCHEE FORMATION	Sand, shell and marl	None		200'
		Pliocene	Hemphill		Open ocean, shallow, far from land	TAMIAMI FORMATION (Top / Bottom)	Marine lime-stone with a high sand content	100'	Confin-ing Beds	250'
		Miocene	Yorktown (?)							
			Alum Bluff			HAWTHORN FORMATION		350'		600'
			Early Miocene			TAMPA LIMESTONE		150'	Floridan Aquifer	750'
		Oligocene	Oligocene		Open ocean far from land	SUWANNEE LIMESTONE		400'		1150'
		Eocene	Jackson		Open, farily shallow sea	OCALA LIMESTONE		200'		1350'

Figure 2

Northeast corner of excavation showing Fill, black Pamlico Sand in solution holes and Talbot Time crossbedded Miami Oolitic Limestone

Figure 3



Macrophotograph at X 14.5 of Miami Oolite (Oolitic Limestone)

Figure 4A

Macrophotograph at X 17 of top Sandy Limestone layer in Fort Thompson Formation

Figure 4B

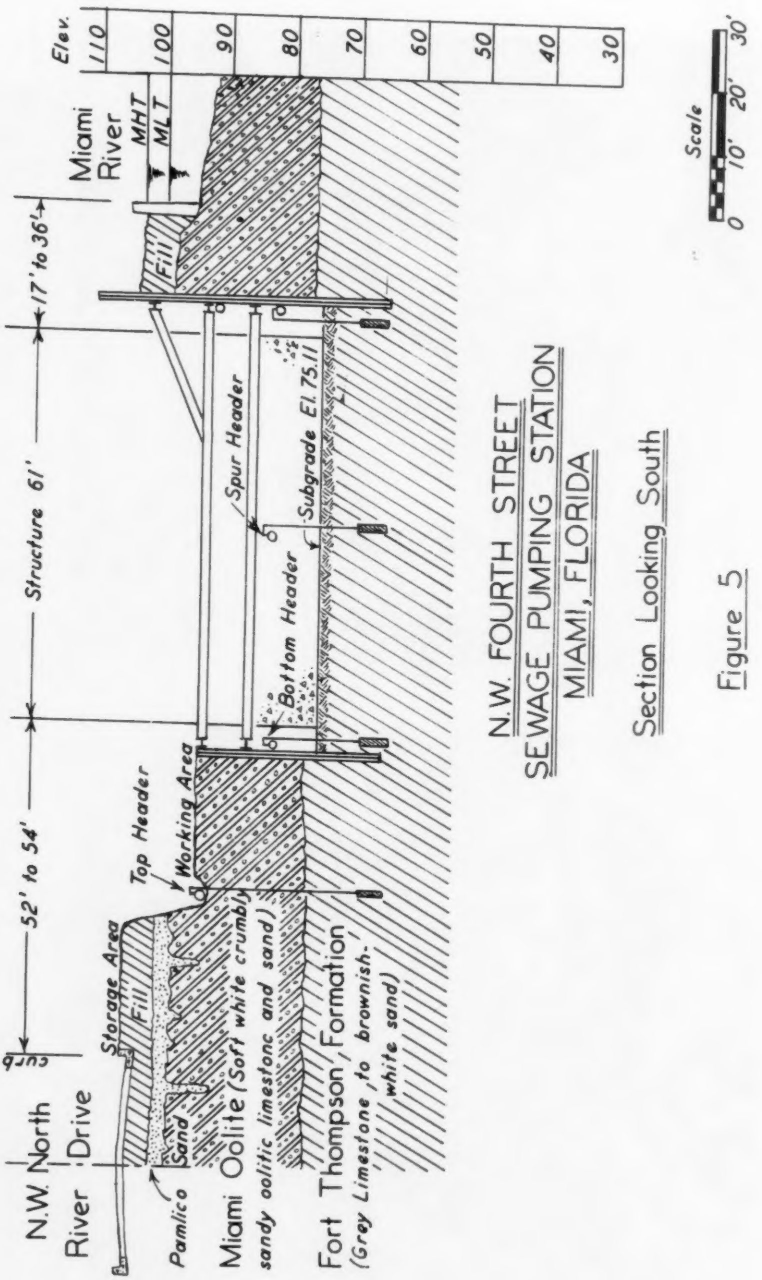

Elev.
110
100
90
80
70
60
50
40
30
Miami River
MHT
MLT
Fill
17' to 36'
Structure 61'
52' to 54'
Spur Header
Subgrade El. 75.11
Bottom Header
Working Area
Top Header
Storage Area
Fill
curb
N.W. North River Drive
Pamlico
Sand
Miami Oolite (Soft white crumbly sandy oolitic limestone and sand)
Fort Thompson Formation (Grey Limestone to brownish-white sand)
Scale
0
10'
20'
30'
N.W. FOURTH STREET
SEWAGE PUMPING STATION
MIAMI, FLORIDA
Section Looking South

Figure 5

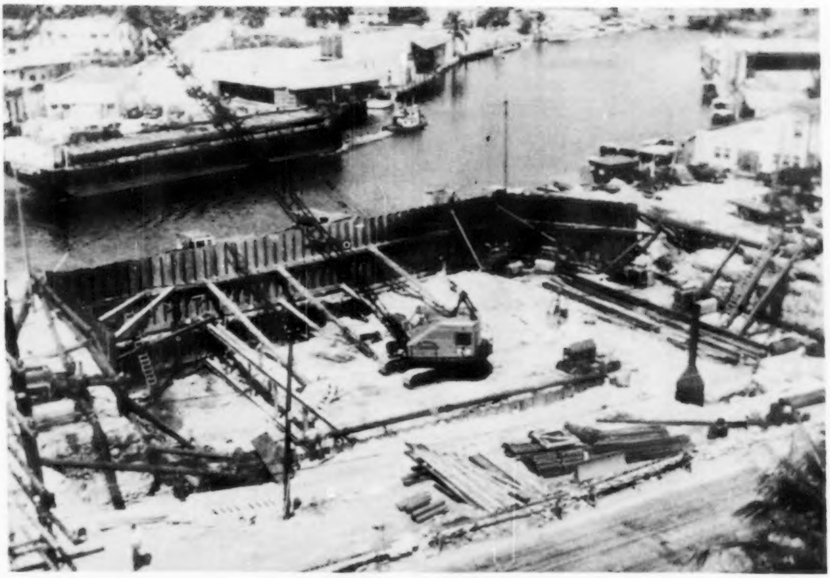


Figure 6A

Diagonal Bracing from Walers at El. 102.0 and El. 94.5

Oolitic Limestone crumbles when excavated for bracing

Figure 6B

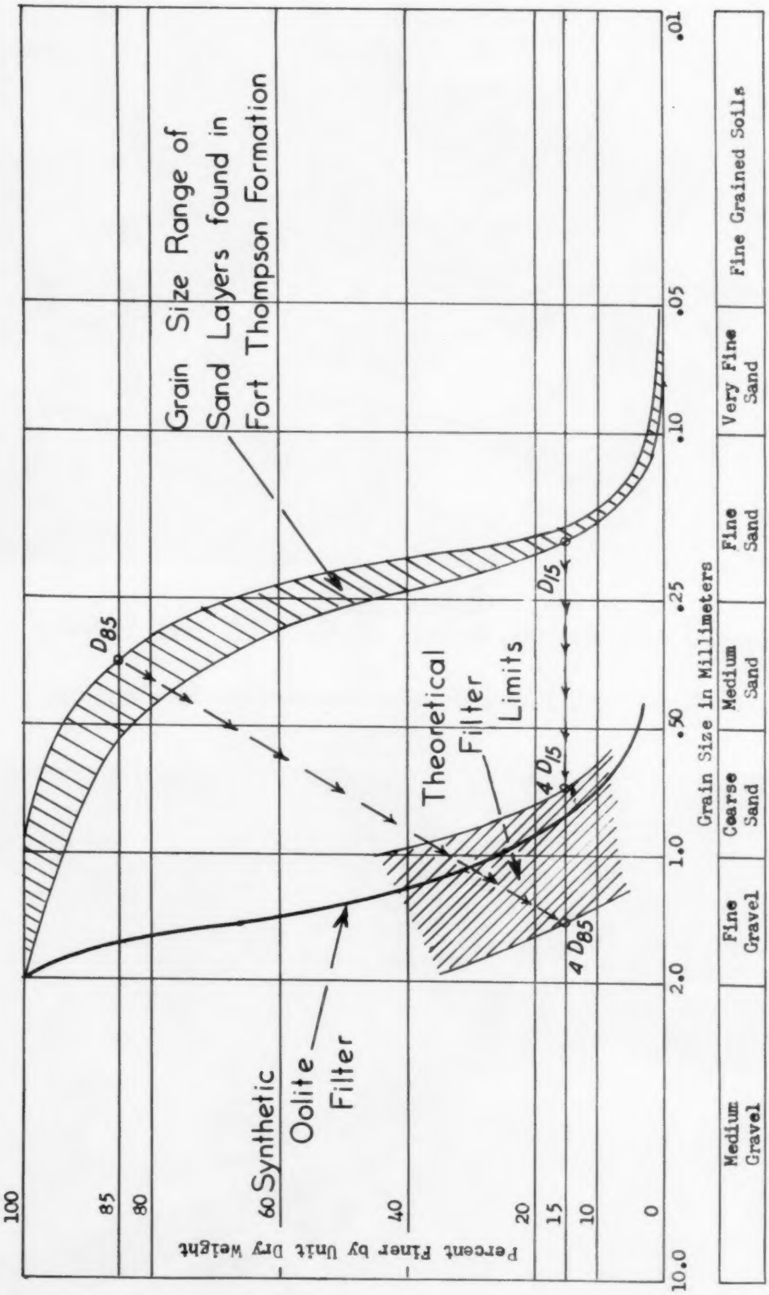
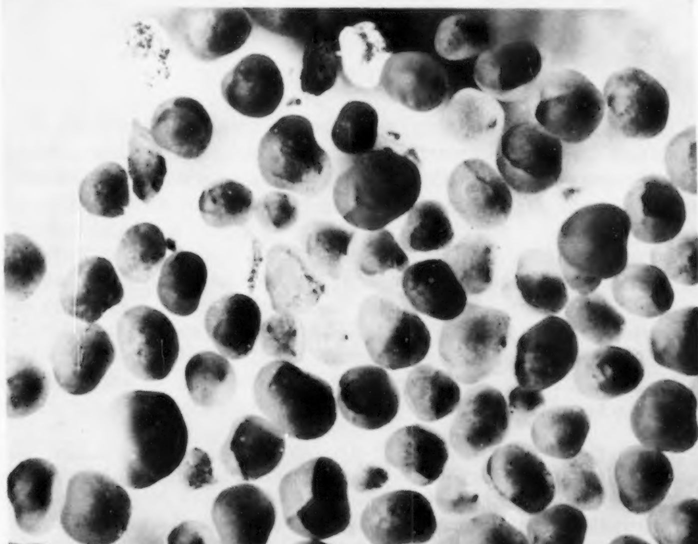

Percent Finer by Unit Dry Weight
100
85
80
60
40
20
15
10
0
10.0
2.0
1.0
.50
.25
.10
.05
.01
Grain Size in Millimeters
Medium Gravel
Fine Gravel
Coarse Sand
Medium Sand
Fine Sand
Very Fine Sand
Fine Grained Soils
Grain Size Range of Sand Layers found in Fort Thompson Formation
Synthetic Oolite Filter
Theoretical Filter Limits
D_{85}
D_{15}
$4 D_{15}$
$4 D_{85}$

Figure 7

Macrophotograph at X 17.5 of angular fine sand in layers in Fort Thompson Formation

Figure 8B



Macrophotograph at X 17.5 of Synthetic Oolite Filter Material

Figure 8A

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

HORIZONTAL DRAINS ON CALIFORNIA HIGHWAYS

T. W. Smith,[1] A.M. ASCE and G. V. Stafford[2]
(Proc. Paper 1300)

SYNOPSIS

With the knowledge gained through the experience and study of the pioneers in the installation of horizontal drains, and the dissemination of this knowledge among the engineers, many agencies both public and private have adopted horizontal drains. Since their use has become so widely accepted it seems timely to discuss the changes in methods of installation, development of equipment, engineering aspects, and the merits of horizontal drains as a means of slide or slipout correction or prevention in the light of California's experience after these sixteen years of elapsed time and thousands of feet of drains installed.

INTRODUCTION

The California Division of Highways installed what is believed to have been the first horizontal drains for slope stabilization in 1939. Basically horizontal drains are holes or borings that are drilled into an embankment or cut face. They are on a slight plus grade and are usually cased with perforated or porous liners.

Since that first job in 1939 approximately 235,000 lineal feet of horizontal drain has been installed by State forces and several installations have been made on certain roads by contract during construction.

Horizontal drains have served a useful purpose in the construction and maintenance of highways in California. They have far more than paid their own way in the saving of construction and maintenance dollars. This word of caution, however: they are not the answer to all problems relating to slides

Note: Discussion open until December 1, 1957. Paper 1300 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83, No. SM 3, July, 1957.

1. Senior Materials and Research Engr., Materials and Research Dept., California Div. of Highways, Sacramento, Calif.
2. Asst. Highway Engr., Materials and Research Dept., California Div. of Highways, Sacramento, Calif.

and slipouts, nor should they be considered a substitute for good planning, design and construction.

Experience has shown that horizontal drains are effective under a wide variety of soils and geologic, topographic, climatic, and ground water conditions. Some drainage installations have been made that were completely effective, a few have been made that were not effective, and the remainder have been effective in various degrees. The majority of installations made by the California State Division of Highways falls in this latter category. Horizontal drains are frequently used in combination with other measures to prevent or correct slides and slipouts.

The use of horizontal drains for prevention or correction of conditions that produce slides can be discussed best under several rather broad headings. These headings are purpose, investigation, planning and installation, equipment, maintenance and effectiveness. The phases naturally overlap and in actual practice cannot be completely segregated.

Purpose

The principal function of horizontal drains is to remove excess subsurface water from hillsides, cut slopes and fills. They are used in an effort to prevent slides by correcting the conditions that cause slides in cut slopes or embankments in certain types of soil or rock formations. They perform this function by providing channels for drainage of subsurface water either from the mass of sliding soil or from its source in the adjacent area.

The removal of the subsurface water tends to produce a more stable condition in several ways. The seepage forces are reduced. These seepage forces are not necessarily in the direction of sliding, but in general they are far more detrimental than beneficial. Removal of the subsurface water tends to increase the shear strength of the soil. Cohesive soils that have a very high shear strength in a relatively dry state may have almost negligible shear strength in a saturated condition. This is especially true of plastic soils in fissures or in planes that have been weakened by previous movement. Removal of the subsurface water reduces any excess hydrostatic pressures that develop. Associated with excess hydrostatic pressures there is a loss in normal forces and hence a loss in frictional shear strength. Thus, a reduction in excess hydrostatic pressure causes a restoration or an increase in the frictional shear strength.

Investigation

The earliest phase of the necessary investigation that should precede the installation of horizontal drains should usually consist of a field review of the site to evaluate the conditions that are causing or tending to cause a slide or slipout. During this phase various methods of correction or treatment are considered. These methods may include horizontal drains. The people making the field review should be competent engineers or geologists who are familiar both with the causes of slides and slipouts, and the various methods of evaluating these causes. They should also have knowledge of various means which might be used to correct or improve the conditions.

The second step in the investigation frequently consists of geologic investigations, and/or exploratory borings, either vertical or horizontal. Generally,

it is advantageous to have vertical boring data prior to the installation of horizontal drains. However, there are many instances where horizontal borings can be used for exploration purposes. In these instances the necessary soil and geologic data may be obtained and, at the same time, the holes may serve the more practical purpose of drainage.

The exploration should provide information on soils and geologic conditions as well as information on ground water conditions. In the installation of horizontal drains, it is important to know the location of the ground water and also the character of the material in which this water might be intercepted.

In vertical borings, once water is encountered, it is frequently very difficult to determine whether the entire soil stratum beneath is saturated or if there are layers of "perched" water. This information is important in installing horizontal drains and frequently it can be obtained by drilling exploratory horizontal holes.

Planning and Installation

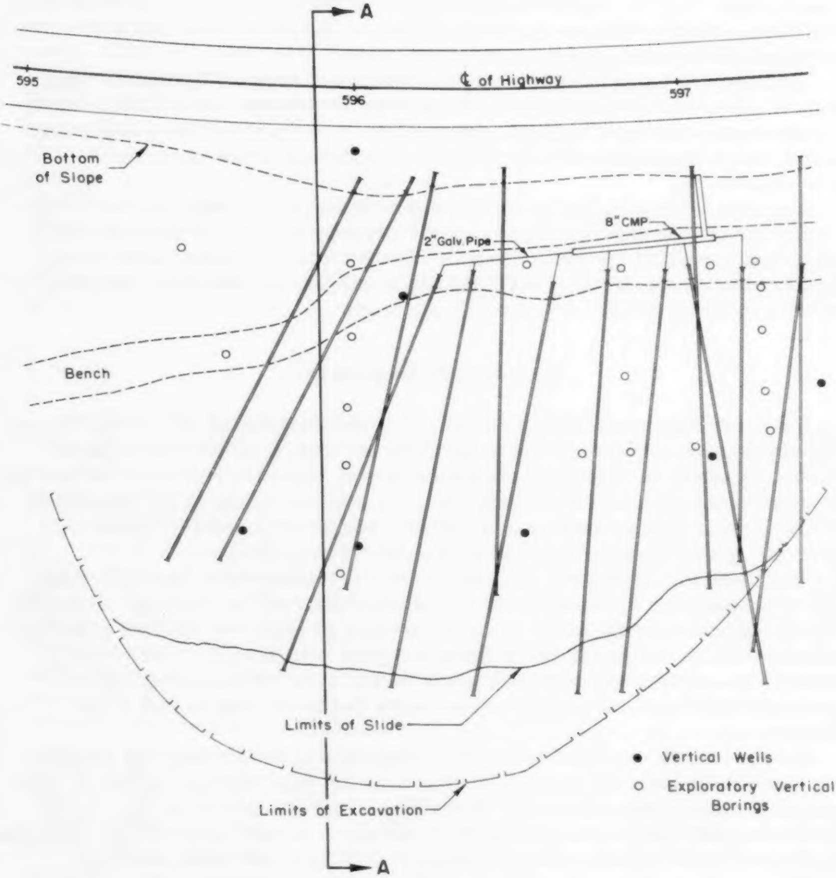
The first step in planning a horizontal drain installation is a careful analysis of the information available from field reviews, geologic investigation, borings, ground water studies, maps—including contours and sections, and all available data on construction and previous soil movement in the immediate vicinity. It should be emphasized that this phase of the work requires experience, sound judgment, and often considerable ingenuity.

The location and depth of the ground water, together with the topography will determine the locations from which the drains will be started. Since the drains will remove the water from the area by gravity, the starting point for a drain must be below the elevation of the point where water is to be intercepted. An exception to this idea would be the relief of excess hydrostatic pressure by a combination of vertical wells and horizontal drains (Figs. 1, 2 & 3).

The spacing of the drains should be dependent upon the drainage characteristics of the soil, the quantity of water intercepted, and the character and magnitude of the slide involved. In drilling from any one elevation, usually drains are planned at intervals of 25 to 100 feet. Drains spaced with intervals of 100 feet would seldom provide adequate drainage, but would determine whether water could be intercepted. If large quantities of water exist it is frequently necessary to space the drains at intervals of less than 25 feet. Drains are often installed from more than one level if the terrain permits and the distances are such that the subsurface water can be reached from various levels.

The depths to which the drains are placed are controlled by several factors. Perhaps the primary factor is the depth to which the drains must extend to contact the water bearing strata and properly drain the area and produce a stable condition in the slide. Other factors that may actually control the depths are difficulty of drilling, quantity of water drained, the economy and effectiveness of a greater number of shorter drains compared with fewer, but longer drains, and occasionally some other factors. The California Division of Highways have installed drains of various depths from 50 to in excess of 300 feet. Most of the drains have been between 100 and 200 feet long.

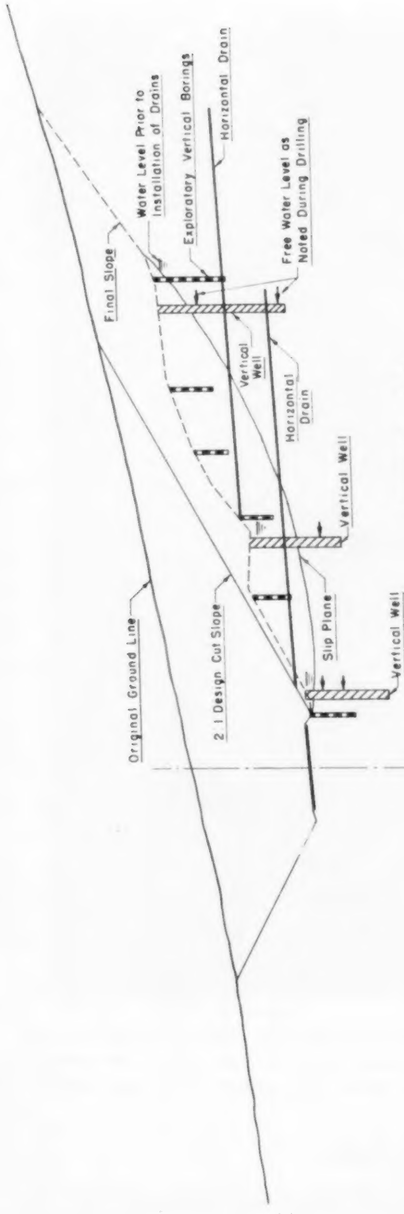
The grades on which the holes are drilled are largely determined by the



SLIDE CORRECTION

HORIZONTAL DRAINS IN COMBINATION WITH VERTICAL DRAIN WELLS

FIG. 1



SECTION A-A

SLIDE CORRECTION

HORIZONTAL DRAINS IN COMBINATION WITH VERTICAL DRAIN WELLS

FIG. 2

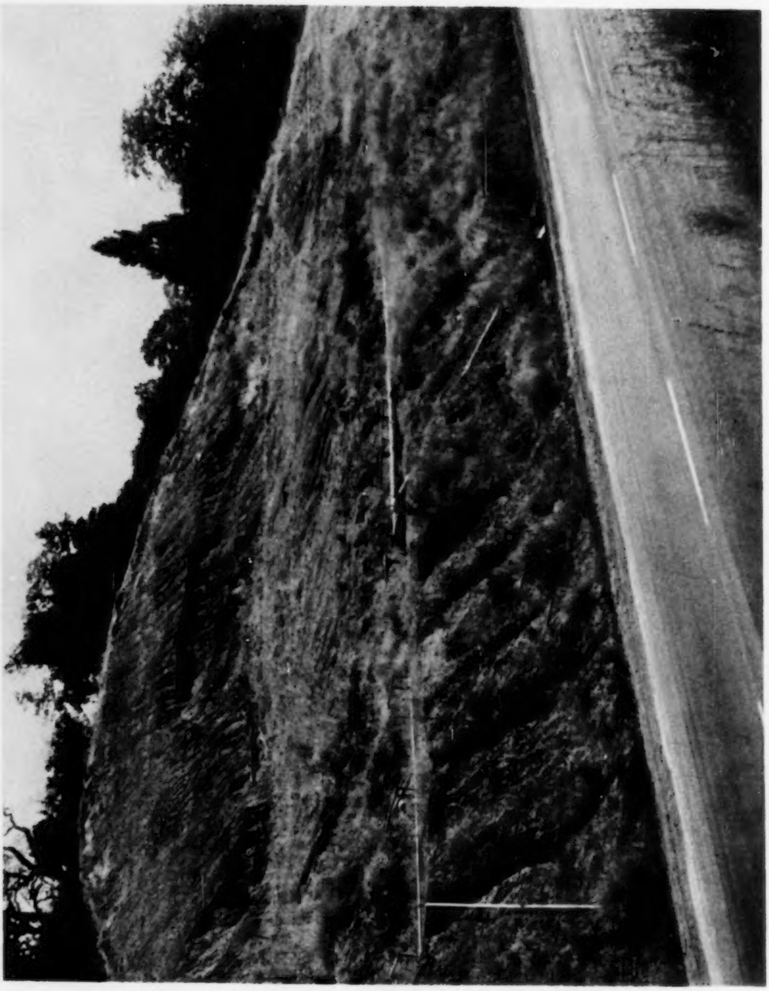


Fig. 3. Slide condition corrected by vertical wells and horizontal drains (See Fig. 1 & 2).

topography and the ground water conditions. The drains must of necessity start from some point that can be reached with the drilling equipment or to which access can be provided. To be serviceable the upper portion of the drain must intercept the subsurface water that is contributing to the unstable soil condition. The drains are usually installed on grades ranging from of 3 to 20 percent with 10 percent being the best working grade. Occasionally steeper or flatter grades are used, but generally they are not practical. There is a tendency to lose or gain grade in the drilling operation. If soil conditions, through which the drilling is done are uniform, the tendency is to lose grade due to weight of the drill rods and bit. Non uniform character of the formation through which the holes are drilled has a great effect on maintenance of grades. The bit will tend to follow the path of least resistance, i.e. to follow fissures, soft layers, or along the contact between layers of material having different drilling characteristics.

Caving has proven to be the greatest single difficulty encountered in the installation of horizontal drains. Slide areas are usually composed largely of loose and broken material and the walls of the hole are unsupported while drilling is in progress. Much footage is lost due to the hole collapsing either during drilling or after drilling has been completed, but before the casing can be installed. With modern rock bits it is possible to drill very hard formations, but no satisfactory method of drilling and casing in one operation has yet been devised.

One other important consideration in planning a drain installation is a collection pipe or system to carry the intercepted water out of the critical area. (Fig. 4). If the outlet of the drains discharge into an existing gutter, usually no other steps are taken. If this is not the case, some other means is used. Access is provided at the lower ends of the drains for future inspection and cleaning.

The most satisfactory installation has been the use of 6" to 8" galvanized corrugated metal pipe. The drain outlets are connected to the larger pipe which in turn collects the water from the drains and carries it to any desired location outside the slide area, such as a natural surface channel.

At least two important things should be kept in mind during layout and installation of the collecting pipes:

1. Easy accessibility of the individual drain for future inspection and maintenance is important.

2. Collecting pipe should be anchored in such a way (preferably to drain outlets themselves) that slight slide movement or local sloughing will not cause the collector pipe to move away from the drain outlet and become disconnected.

Where freezing may occur during the winter season it may be necessary to bury the collecting systems.

Open flumes and paved ditches have been used for collecting and carrying the intercepted water, but because these two types of collectors require constant inspection and cleaning they have not proven too satisfactory.

Equipment

The first equipment used by the Division of Highways in 1939 for horizontal drilling was relatively light. (Fig. 5). It was originally designed and

Fig. 4. Completed drain installation and collector pipe system.

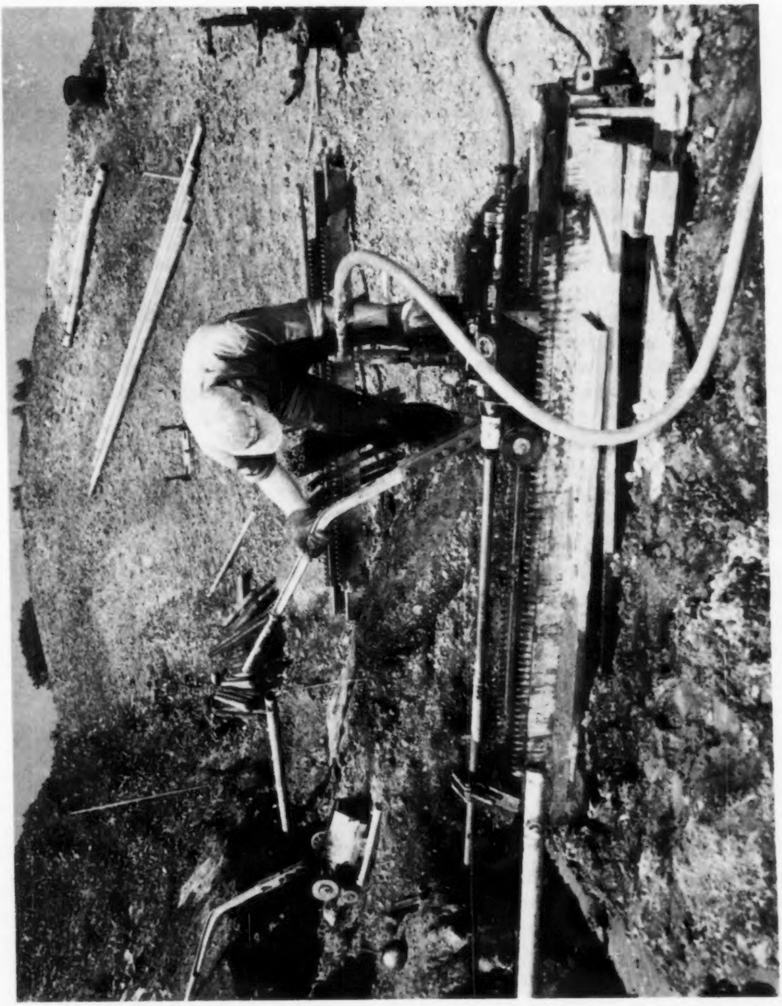


Fig. 5. Operating Hydrauger equipment.

developed for utility construction work where insertion of pipes under side walks, streets, highways and property was necessary without disturbing the surface.

This equipment consists of a rotary drill mounted on a racked frame in such a way that a revolving drill bit may be advanced into the earth with a hand operated ratchet lever while water is pumped through the drill road to cool the bit and wash the cuttings from the borings. (Fig. 6). Five-foot lengths of drill rod are added as the drilling proceeds. Compressed air engines which are quite compact and portable are utilized for pumping water and operating the drills. The compressor and water pump are placed at convenient locations with suitable pipe or hose lines leading to the drilling units. With this arrangement of equipment it is not necessary to move the heavier compressor and pump when moving from the location of one boring to another.

The first horizontal drain holes were drilled by a 2" pilot bit and then reamed to 6" in diameter before casing with 4" perforated metal pipe. It was soon found that it would be more practical to perform the drilling in one operation. This resulted in the adoption of a 4" modified fishtail bit to do the drilling in one operation.

The 4" modified fishtail bit was progressively improved, beginning with construction out of a good steel followed by facing on the cutting edges with tube borium. The ultimate was reached with this type of bit by using a combination of tungsten carbide inserts and tube borium on the cutting and wearing surfaces for drilling in hard formations.

Various other types of commercially available bits were tried. No better bit was found until 1949 when the rock roller bit commonly used in the oil fields became available in small sizes. (Fig. 7). These bits were tried, proved superior in all formations except possibly stiff plastic clays, and have been used almost exclusively since.

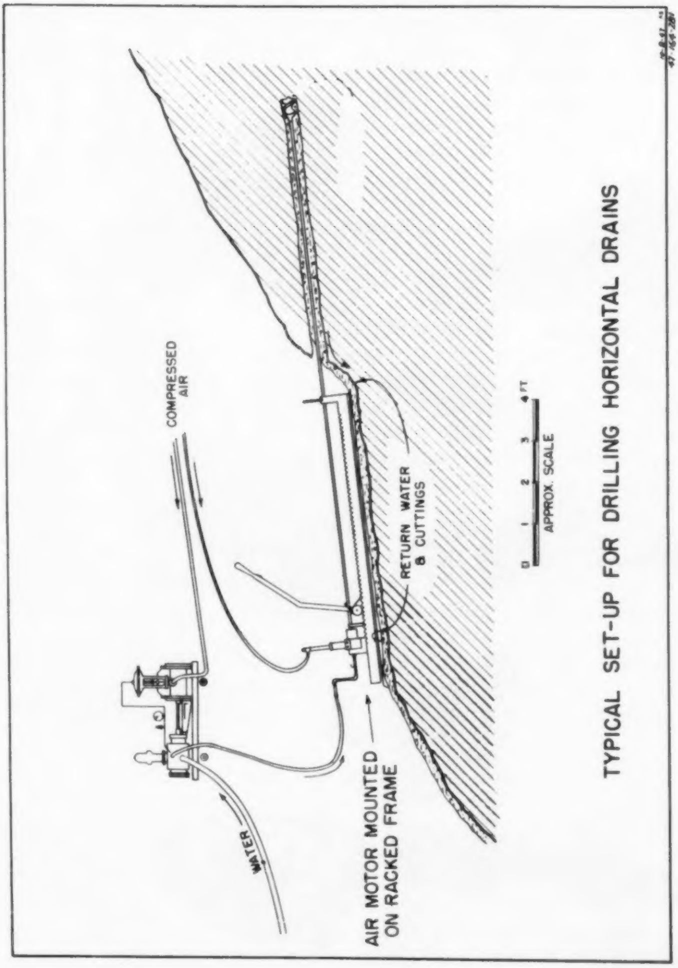
Standard perforated 2" iron pipe with the following specifications is used for casing:

> "Standard 2" black steel pipe perforated with 3/8" diameter holes on 3" spacing drilled in 3 rows at the quarter points, to be furnished in random lengths of 16 to 24 feet without threads or couplings. Pipe to be vertically dipped in a standard pipe dipping asphalt subsequent to drilling."

This casing is butt welded with oxy-acetylene equipment as it is jacked into the hole. The principal purpose of welding the joints together rather than using screw joint couplings is to hold the perforation rows in alignment. The perforations are normally placed up so the intercepted water will be carried out of the critical area to discharge into the collector drain.

The borings are cased by the use of an extra carriage which replaces the drill on the racked frame after the drilling is completed. The casing was originally held in the carriage saddle and kept from slipping by the use of 36" pipe wrenches. This arrangement was later revised and improved by constructing a grip similar to a standard chain pipe vise as an integral part of the carriage.

During the first few years of drain installation work a sharpened wooden plug was driven into the leading portion of casing to serve as a guide with the idea in mind that a retractable folding bit could be used through the casing to drill out an occasional large obstruction caused by caving during the casing operation. Experience indicated that in most cases the wooden plug was



TYPICAL SET-UP FOR DRILLING HORIZONTAL DRAINS

FIG. 6

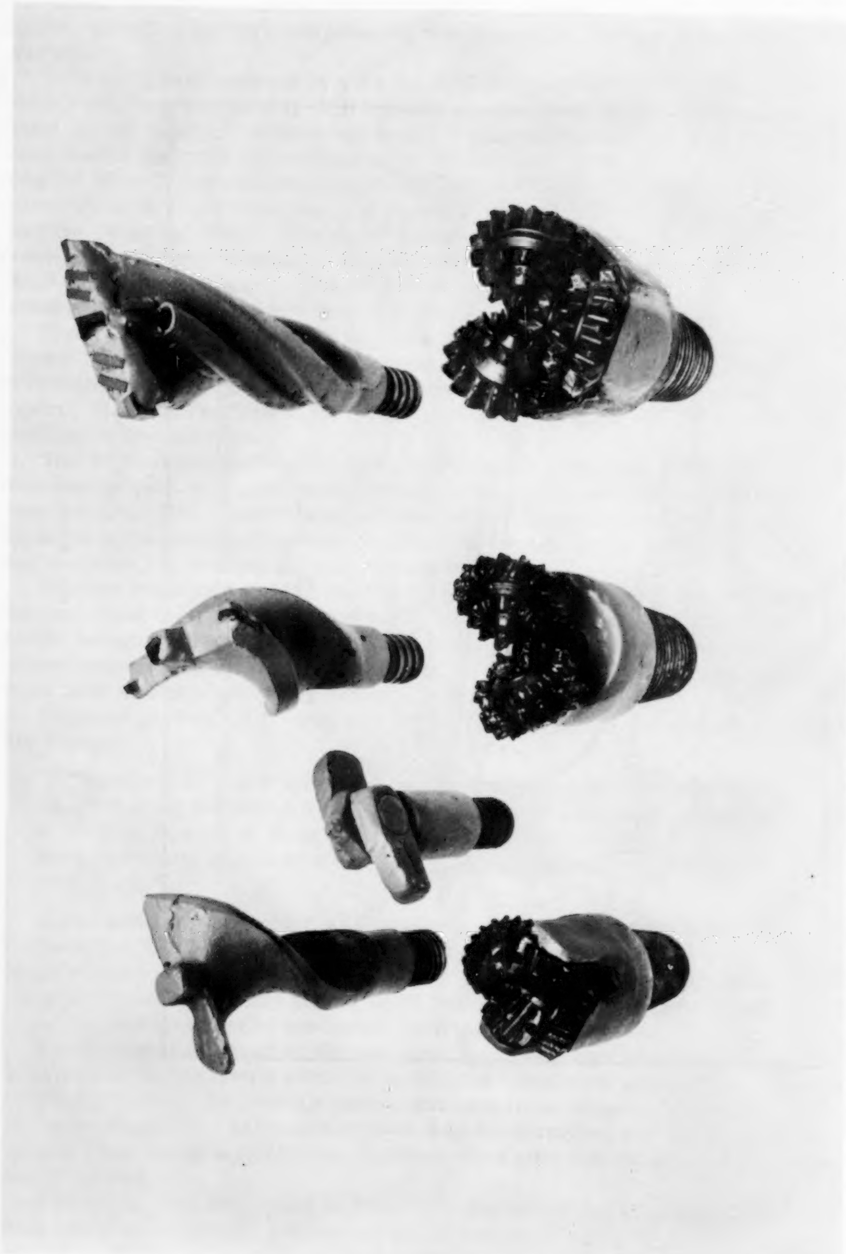


Fig. 7. Bit types past and present used for drilling horizontal drains.

merely driven back into the casing and only served the purpose of plugging the end. This did keep soil and rocks from entering the casing but drilling out cave-ins did not prove too satisfactory. This procedure was abandoned in favor of pointing the perforated metal pipe itself and rotating, occasionally, to push obstructions out of the way.

With the experience and progress in horizontal drilling technique it was recognized that more powerful drilling equipment should be obtained to supplement the lighter equipment in use. Consequently a more powerful machine with a 60 horsepower gasoline engine was purchased early in 1951.

This drill is a self-propelled unit, capable of moving about within limits of a job, having a hydraulic feed, using continuous flight augers and requiring no water for drilling. (Fig. 8). It proved to be a very good rugged piece of equipment, but the continuous flight augers were limited to drilling in soil or soft rock formations. The practical drilling depth, due to lack of directional control caused by the necessary flight coupling arrangement, limitations on power, and strain on the equipment proved to be only about 150 ft. in the formations found in slide areas.

In 1953, two years after obtaining this more powerful machine, accessory equipment was fabricated so that regular rotary drilling could be accomplished using diamond drill N-rod and 4-1/2 in. rock roller bits. (Fig. 9). The degree of success upon converting the machine to a rotary type drill led almost immediately to the existing phase of equipment development.

The converted machine operated very satisfactorily, and it was found that the use of heavier drill rod, the hydraulic feed and superior power all were advantageous. This machine, however, had one serious drawback: when using the machine for forcing the casing into the drilled hole, the casing must be in front of the drill carriage, as the design of the machine prevents working through a chuck; this necessitates using lengths of casing which can be inserted between the carriage and outlet end of the drain at the ground surface. In restricted working areas it is necessary to use 5 ft. lengths of casing, with a correspondingly large number of field welds. Also this machine was somewhat larger and more powerful than necessary for the rotary drilling work on drain installations, which results in some sacrifice in mobility.

Development of Improved Machine

No drill rig on the market designed specifically for drilling holes for horizontal drains could be found and none which satisfied California's requirements. A drill rig was desired incorporating the following features: the drill unit should be complete with a gasoline engine of adequate power; a suitable transmission to permit control of speed of rotation over a wide range; a hydraulic feed with a minimum stroke of six feet, capable of exerting a 4000-lb. thrust; provision for slowly rotating the casing concurrently with the jacking operation when necessary; a chuck readily interchangeable for A-rod, N-rod or casing and so designed that long lengths of rod or casing can be operated through the chuck; rugged, but easily operated spuds for maintaining alignment and grade of the drill; rubber-tired wheels and three-point suspension to permit sharp turns; and, finally the overall length not to exceed twelve feet and the weight of the complete drill to be not more than 3000 lbs.

The California Division of Highways Materials and Research Department had for several years realized the need for such an improved horizontal drill,

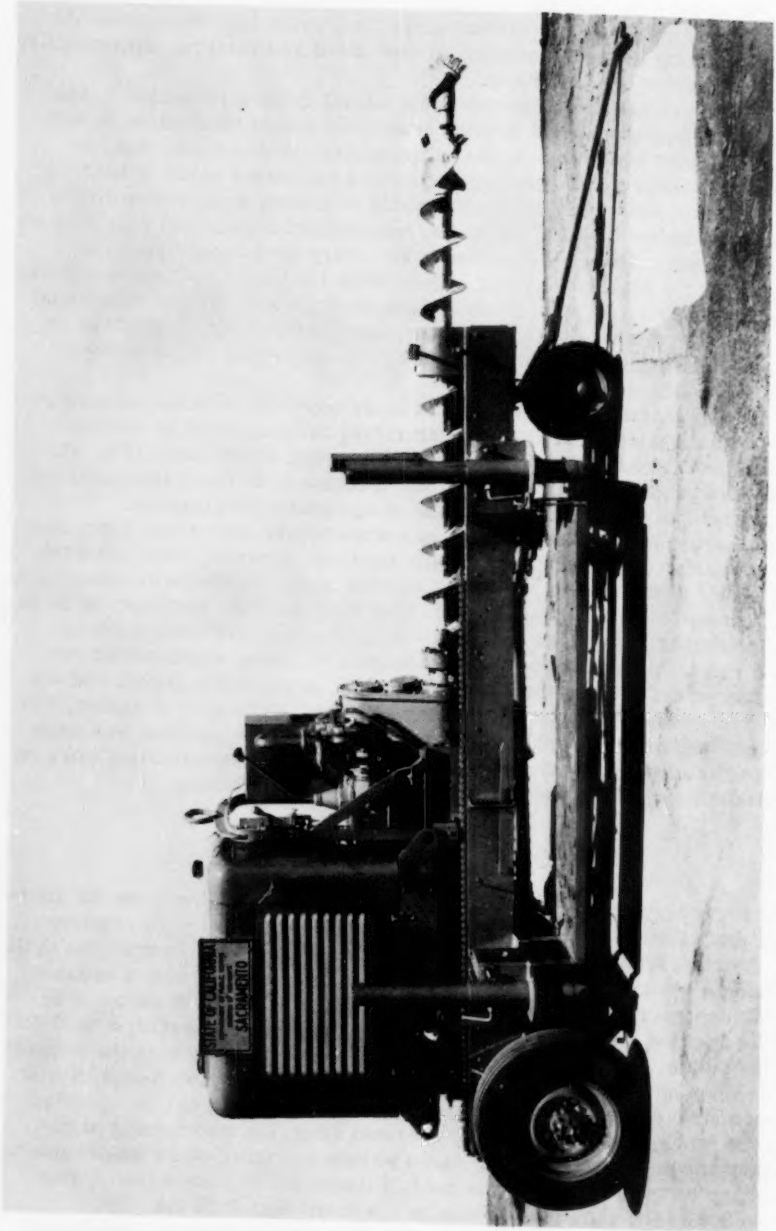


Fig. 8. McCarthy Rock Boring Machine with continuous flight augers.

Fig. 9. McCarthy Rock Boring Machine modified to drill with circulating fluid.

and as no completely suitable machine could be purchased, it was decided to design and build a drill unit specifically for horizontal drilling. The new drill rig, (Fig. 10) for the most part, is comprised of standard or proven parts of sub-assemblies similar to those used in manufactured drills. The machine is unique because it incorporates the desirable features of various machines into a light-weight, compact drill rig especially suitable for the type of drilling required for installation of horizontal drains. The power unit is a 20 horsepower 4-cylinder, air-cooled engine, connected through a fluid drive to a 4-speed transmission. Rotation of the chuck is accomplished by a gear train from the transmission enclosed in an oil-tight housing. The entire drive assembly is mounted on a hydraulically operated carriage with a travel of six feet. A ten-gallon-per-minute oil pump, also driven by the 20-horsepower air cooled engine, supplies oil to two hydraulic cylinders, by means of which the thrust can be controlled at any desired feed pressure up to 4000 lbs.

A specially designed chuck assembly was required to permit the use of long lengths of drill rod or casing, and to provide for interchanging chucks for different size rods. Standard A-rod and N-rod chuck heads are used, with a shop-designed chuck holder which permits quick change of chuck heads. A special chuck for gripping the two-inch casing is used in the same chuck holder.

While the horizontal drilling equipment has gone through several stages of development, the development has usually resulted from particular needs dictated by drilling conditions encountered. This is borne out by the fact that while the new California Horizontal Drill just recently built to particular specifications, which make it definitely superior in many respects to other available equipment, several of the original units are still being used essentially as they were modified in 1939 for the first horizontal drilling. These light, portable units still have the advantage of greater mobility where access and set-up room is a problem.

Maintenance

Some maintenance of horizontal drain installations is necessary if they are to continue to be effective for long periods of time. The maintenance required is dependent upon local soil conditions, vegetation, rainfall, road conditions, and other factors.

Considerable maintenance has been eliminated by the use of approximately 20 lineal feet of non-perforated galvanized pipe for the last length of casing placed in the boring. This, in most cases, minimizes trouble from root growth in the casing and retards corrosion at the outlets. (Fig. 11). Aside from regular inspection and repair of visible damage and obstructions the major maintenance consists of cleaning at intervals of three to ten years to remove root growth, corrosion, and soil from drains.

It is important that the drains be cleaned thoroughly when their efficiency has been impaired. This, in the great majority of cases, restores them to their original effectiveness.

While limited cleaning may be done with hand tools the only satisfactory method has been with the use of power equipment. This is usually accomplished with the original installation equipment modified for this purpose or with a relatively light hand-held air motor similar in operation to the original horizontal drain machine. Diamond drill E-rod with an auger type bit small

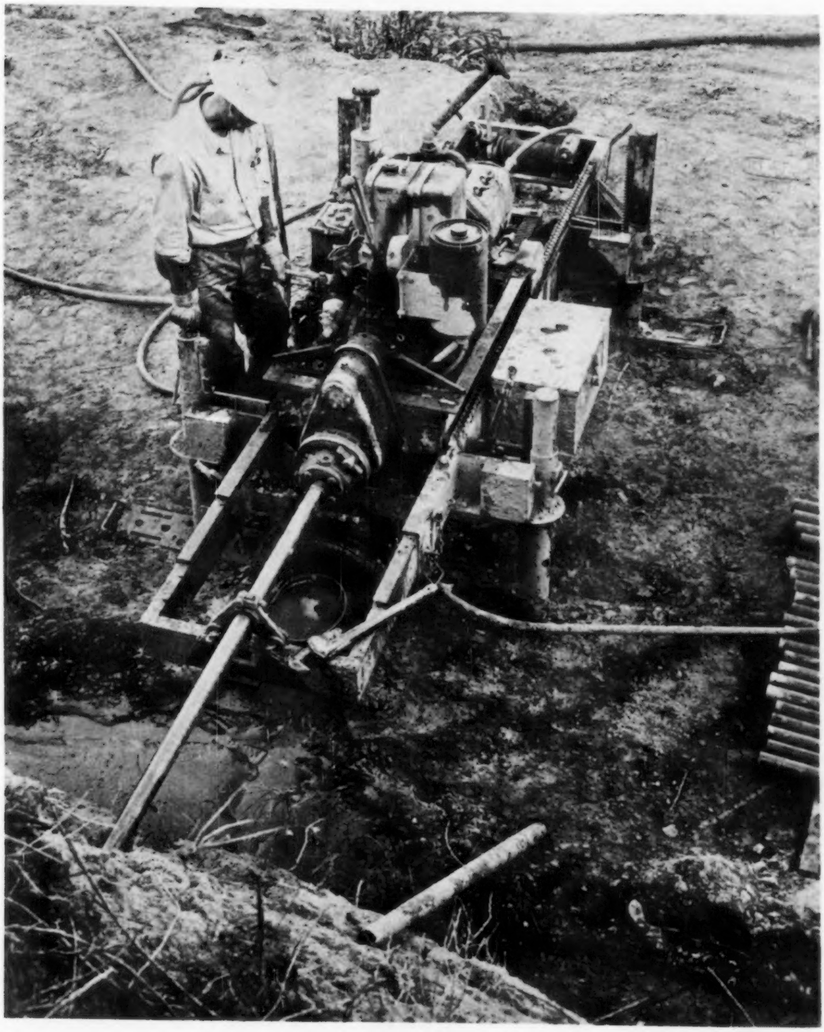


Fig. 10. California horizontal drill rig in operation.

Fig. 11. Extreme case of root growth in horizontal drain.

enough to rotate freely in the casing is used for the drilling tools. Some of the water jets of the cleaning bits are so directed that the casing is thoroughly washed and flushed as the bit progresses through the casing. Water delivered to the air motor at 150 psi to 200 psi has proven adequate. Standard practice has been to clean the drains to a 50 ft. minimum depth or 10 ft. beyond the point where the flushing water becomes clear when greater cleaning depths are necessary.

The chief advantages of machine cleaning operations are:

1. Drains may be cleaned to much greater depths than by hand.

2. Cutting out the root growth entering the casing perforations is more complete.

3. Vibration as a result of machine operation of cleaning tools combined with adequate pressure and volume of the flushing water tend to loosen and wash out rust, scale, soil and any other deleterious material which may be in the casing or its perforations.

4. Vibration also tends to agitate soil adjacent to the casing, allowing larger flow of subsurface water into casing.

It has sometimes been necessary to replace the outlet ends of the casing where rusting and corrosion have caused excessive damage to the "stickout." Galvanized pipe is always used for this replacement. No replacement to date of this portion of the drain has been necessary where a length of galvanized pipe was used for the outlet end in the original installation.

Effectiveness

Drain installations that have been preceded by adequate investigation and that are properly planned have usually been effective in removing subsurface water and this in turn has corrected or improved the unstable condition.

Drain installations have been most effective in areas where the subsurface water could be intercepted in well defined aquifers or layers, where the soil was sufficiently permeable to permit ready removal of the water, and where the water could be reached with holes not more than 300 feet long on 5 to 15 percent grades through formations that can be drilled successfully and where the borings do not cave.

Some installations made with as few as eight to ten drains, all less than 100 feet in length, have been effective in correcting a slide. Other installations have required in excess of one hundred drains, some of which were drilled to the maximum practical depth.

Perhaps the two types of foundations in which it is most difficult to make installations are (1) silty fine sands that tend to erode or wash badly during the drilling operation and cave to the extent that casing the holes is difficult; and (2) hard broken formations that are difficult to drill. In this type of material loss of circulating fluid is a problem, as well as the caving during drilling and casing operations. Competent drilling personnel are a must in the installation of drains and this is certainly emphasized when drilling or casing operations are difficult.

The quantity of water that is produced or drained at the time of installation may not be a good measure of the flow that will occur later or of the effectiveness of the installation. Some drains may produce large flows during the

rainy season or during and after actual periods of rainfall, and be dry or produce very little water at other times. Other drains may produce flows that vary somewhat with the seasons yet are relatively constant. It is also true that in some instances the removal of a relatively small quantity of subsurface water will produce a stable soil condition, whereas other instances may require the removal of very large quantities of water to produce the desired results.

Discussion of Representative Installations

Lookout Point

This is a side hill cut adjacent to the Pacific Ocean on U. S. Highway 101 approximately 1 mile south of Orick constructed in 1949. The toe of the cut is approximately 1000 ft. long at roadway level tapering to about 100 feet in length at the top of the cut some 225 ft. above the road. Traffic during construction was carried by the existing road at the top of cut and this road would have been endangered by a major slide. An echelon faulting with many slip planes and old landslides were in evidence particularly in the northern portion of the area. The cut was constructed on a 1:1 slope with benches 60 feet apart in elevation. The material encountered consisted chiefly of a graphitic schist interbedded with seams of fractured quartz.

During the early stages of the excavation of the cut active springs were exposed on the cut face. It was decided during this phase of construction that a few exploratory horizontal borings should be made to locate the water bearing formations and to determine the feasibility of effectively subdraining the entire cut area by this method. An initial flow rate of 10,000 gallons per day was developed by the first 4 exploratory drains. From this and the excavation data it was soon apparent that the bulk of the subsurface water was located in the lower two-thirds of the cut slope and that a comparatively large volume of this seepage could easily be intercepted and controlled by means of horizontal drains.

The horizontal drain installation was continued, covering the cut slope area from the two lower benches and roadway level with a total of 30 drains approximately normal to the highway centerline. These drains averaged 127 feet in depth. The horizontal holes ranged in grades from 6 to 22 percent. The total maximum flow rate from all the drains amounted to over 140,000 gallons per day which decreased to 5,000 gallons per day after the perched or impounded water was drained out.

The installation continues to produce around 5000 gallons per day during the wet season of the year decreasing to only a few hundred gallons per day during the summer months.

At this location the horizontal drains were installed as a preventive measure in an unstable area where the slide potential was definitely aggravated by the presence of a considerable amount of ground water. The conceivable magnitude of a slide in such a large cut warranted any reasonable stability insurance that appeared practical.

Six years have elapsed since construction. Several million gallons of water have been drained out of this potentially unstable cut during this period. Only minor sloughing and local slides have occurred since construction, and it appears that the horizontal drain installation has been effective.

Willits Slide

This slide occurred in the right side of a thorough cut which was constructed in 1947. This cut is 2-1/2 miles south of Willits on U. S. Highway 101. Seepage was noted in the cut slope during construction, but was not alarming in view of the planned slopes of 2:1. However, in the winter of 1948 three small slides developed and were smoothed up by maintenance forces. During the winter of 1950 the slide again became active, probably because of the exceptionally heavy rainfall that year. The amount of seepage water was much greater than previously observed and the sliding was more extensive. A break joining the three previous slides developed much higher on the slope and a plastic movement of the entire mass created a traffic hazard by encroaching on the traveled way every winter until the installation of 17 horizontal drains in 1953. See Figs. 1, 2 and 3.

When the horizontal drains were installed the upper limit of the semicircular slide scarp extended about 220 feet from the highway centerline and the slide was about 300 feet wide at the toe of the slope. A stratum of stiff blue clay composed the first 20 feet of the cut slope and had remained fairly stable through the years. This blue clay was overlain by alternate layers of brown silty clay and terrace gravel, which were sliding over the blue clay and onto the traveled way.

One unique feature of this installation was the placing of several vertical borings near the outer periphery of the slide scarp to connect the pervious gravel strata separated by the impervious clay.

Ten of the 17 horizontal drains were installed along the contact of the blue clay and overlying material, while the remaining seven drains were installed from roadway grade. In general the drains were installed from the northern two-thirds of the slide and all are angled slightly toward the south. The upper drains were spaced 10 to 20 feet apart while the roadway drains vary from 10 to 50 feet. The horizontal drains averaged 130 feet in length and produced a maximum initial total flow of 13,265 gallons per day. Of this total, one drain located near the south extremity of the slide produced a flow of 10,000 gallons per day. These initial flows decreased as the various water forces in the sliding mass were relieved.

The sustained flow from this installation has never been very high, ranging from a relatively small amount during the summer to something in the order of 1500 gallons per day during the winter. However, this relatively small daily flow does amount to an appreciable quantity of water when added up over a period of several months. At least the sub-surface drainage of the slide area has been sufficient to stabilize the slide, for only local surface sloughing has occurred during the three winters since the horizontal drains were installed, while sliding has taken place in adjacent areas not stabilized by horizontal drains.

Sears Point Slide

This slide is located on the Black Point Cutoff road north of San Francisco Bay between Vallejo and highway 101. Several major earth movements occurred at this one location during construction and at least two of these repetitive slides reached the median of the newly graded four-lane roadway. After about 80,000 cubic yards of slide material had been excavated from the area 10 horizontal drains, totaling 2000 lineal feet, were installed from one level in the approximate center of the slide. Immediately after the installation of

these drains another major slide occurred which destroyed one drain and damaged several of the others. After the loose material had been excavated and the area again resloped, additional horizontal drains were installed from various levels through the slide area. This is illustrated by Figure 4. Forty-nine drains were installed in this area to an average depth of 157 feet. The 7397 lineal feet of horizontal drain installed in this area makes this the second largest installation ever constructed by this department.

The slide material was composed largely of a soil of a clayey nature which was very unstable in the presence of water and yet was impervious enough to prevent adequate movement of ground water to allow easy subdrainage. This impervious nature of the soil accounts for the high footage of horizontal drains which were required to properly control the subsurface water. Vertical observation wells drilled in the area prove the horizontal drains have effectively lowered the water table in the slide area.

The slide extended about 300 feet along the roadway while the upper limits of the excavation was normal to and about 400 feet from centerline. The final slope of the excavated area was about 3:1 indicating the flatness of the plane upon which this material was sliding. Exceptionally high initial flows were not developed by any of these drains although the combined total was approximately 50,000 gallons per day. Since the water table has equalized at its lower elevation this installation has produced some 4000 to 5000 gallons per day during the summer and up to 20,000 gallons per day during the winter.

Horizontal drains alone could not have been relied upon to control this slide. However, the drains coupled with the unloading and slope flattening have proven to be a successful stabilization treatment. Sliding in adjacent areas continues on very flat slopes. It is believed that the horizontal drains have been an aid in preventing movement in this old slide crater where no sliding has occurred since 1951.

Half Moon Bay Slipout

Failure of this highway embankment took place during the 1951-52 wet season. The highway affected is a 5 mile connecting road in the San Francisco penninsula area connecting State Sign Route No. 5 with State Sign Route No. 1 at the little town of Half Moon Bay.

During the early part of the winter settlement of approximately 200 feet of roadway took place at a point about 4 miles east of Half Moon Bay where a side hill fill existed. This was followed by major movement in the latter part of the winter involving the entire width of the traveled way. A vertical displacement of some 10 to 12 feet with a maximum horizontal movement of slightly less had occurred at roadway level and on the upper part of the embankment. Some movement was indicated about 100 feet down the fill slope.

Exploratory borings showed a large quantity of water in a sand and gravel blanket which had been placed under the embankment about 20 to 25 feet below the outside shoulder. The presence of this water was considered proof of ineffective drainage. Considerable spring water from the original ground slope above the embankment was also indicative of inadequate subsurface drainage.

After analysis of the data horizontal drains appeared to be the only practical means of subdraining the slipout area.

Twenty-six horizontal drains were placed in two general locations. Twelve were placed near the toe of the fill some 40 feet below the road. The other 14

drains were installed from roadway level to intercept the subsurface water before it reached the critical fill area.

Grades ranging from 8 to 25 percent were used for the borings below roadway level. These grades were determined for each succeeding drain as the work progressed by analysis of the boring records of all previous drilling. Those installed from roadway level were drilled as flat as practical. Grades of 2 to 11 percent were used. These grades were largely predicated on data obtained as the drilling progressed. In other words as the various formations were encountered the grades on the next holes were adjusted accordingly.

An attempt was made to install the drains below the road to a depth of 200 feet and those above the road to 150 feet. It was possible to case only about 85 percent of the 4000 feet of drilled hole due to the loose rocky nature of the soil in the area. Caving also made the drilling operations quite difficult.

Maximum initial flows from all the drains aggregated some 13,000 gallons per day. Flow readings taken since the installation was completed show a range from a few hundred gallons per day in the dry summer months to about 8000 gallons per day during the wet season.

The roadway was reconstructed and brought up to grade after the horizontal drains were completed. During this reconstruction an underdrain was installed along the upper side of the roadway for the entire length of the slipout area. This was to intercept subsurface water entering the roadway section near grade.

Four years have elapsed since this work was done. To date no distress or movement has taken place in the area, indicating that the stabilizing treatment, which was primarily installation of horizontal drains, has been successful.

Nevada City Slide

In realigning State Sign Route 20 in the vicinity of Nevada City it was necessary to construct a new intersection with a county road known locally as the Bloomfield connection.

It was necessary to make a side hill cut about 200 feet in length with a maximum height of approximately 35 feet. This excavation was completed during the summer of 1953. The soil encountered was principally a silty sand with varying amount of clay binder. During the winter of 1953-54 severe erosion and gullying of the cut slope occurred, together with some minor sloughing and local sliding.

In May and June a bench about 20 feet wide was excavated approximately 10 feet below the original top of cut, the excavation from this bench was pushed over the cut slope to fill up the erosion gullies and restore a uniform slope.

Upon completion of this resloping, seepage developed in the cut slope below the bench with local sliding reoccurring at the same location as previously. This sliding was of somewhat larger magnitude, with wet slide material moving onto the roadway. Cracking and bulging was in evidence on the cut slope for a distance of about 150 feet along centerline of the road connection. Considerable seepage was evident throughout the broken area. Only about two or three thousand yards were involved at this time. It appeared that the slide would grow progressively worse and ultimately move beyond the slope line. This would create right-of-way problems and would jeopardize a water main just outside of the right-of-way.

Three methods of control were considered, as follows:

1. Removal of the loose slide material, construction of benches, and flattening of the slope; however, with the original slope being 1-1/2:1, this indicated that a very flat slope might be required to stabilize the cut. With this method aquisition of additional right-of-way would be needed and the water main would have to be replaced. This appeared to be too costly.

2. Removal of the displaced slide material and covering the exposed wet slope with a heavy blanket of freedraining rock or gravel. This treatment would have been quite costly due to the necessity of using either a clamshell or dragline rig for placing the pervious material. No suitable pervious material was available with the project limits or in close proximity thereto. Importation of such material would be costly.

3. Subdraining the area by the installation of horizontal drains. Estimates indicated that such treatment would cost about $2000. To stay within the right-of-way the drains could not exceed 100 feet in length.

This latter method of subdraining the area with the installation of horizontal drains was chosen as treatment by the engineers reviewing the problem. It was believed that, although not a positive corrective measure, it would offer reasonable assurance of controlling the slide at a minimum cost.

Twelve drains were placed from roadway grade to depths averaging about 80 feet. Grades varied from plus 15 to 20 percent. The average maximum initial flow rate for the twelve drains was 890 gallons per day.

Drying up of the spring areas on the face of the slide area occurred as the impounded ground water was intercepted by the progressive installation of the drains. By the time the work was completed the flow rate from the completed installation was only 2000 gallons per day. This rate has been typical of the flow during the wet seasons.

Total cost of the treatment amounted to $2350.00 which gave a unit cost figure of less than $2.50 per foot of installed drain.

As of this date no further slide movement of any consequence has taken place in the area, indicating that the treatment has been effective and was well chosen.

Santa Rosa Slide

During construction of the Santa Rosa by-pass in 1947 on U. S. Sign Route 101 it was necessary to make a small cut at the junction with the old highway at the north end of the city. This realignment involved cutting back the point of a ridge composed of a very silty sandy material. The maximum height of the cut was only approximately 20 feet and about 200 feet in length.

During the wet season immediately after construction minor sliding developed in the cut area. During the succeeding winters the sliding continued making it necessary to remove small amounts of material which slide onto the roadway. Since the slide was growing progressively worse during each wet season and was affecting property outside the right-of-way, it was decided that some steps to correct the condition should be undertaken.

Vertical borings were made in the slide area. Data from these borings together with seepage on the surface of the slide indicated that a considerable quantity of ground water was present.

Horizontal drains appeared to be the most logical and economical means of draining the critical area.

Thirteen horizontal drains were installed from two general locations.

Five drains were fanned into the slide area from the north edge of the active slide. The remaining 8 drains were fanned into the area from the south edge of the slide.

Far more difficulty was encountered in installing these drains than had been anticipated. Quantities of sandy material which continually sloughed and caved, blocked the holes so the casing could not be advanced in many cases, resulting in the loss of nearly one-quarter of the hole drilled.

A total of 1645 feet of horizontal drain was installed with 1575 feet of casing being placed in 2078 feet of drilled hole. The drains developed a combined maximum initial flow of about 5000 gallons per day.

The flows produced by these drains were not as large as had been anticipated and the water levels in the vertical borings were not greatly lowered. Some parts of the slide area where it was desirable to place drains could not be reached due to the sandy material mentioned above. The drains installed did not produce any flows which continued in large enough volume to lower the water table in the area.

The advisability of placing the drain perforations down instead of in their normally upward position was considered, due to the possibility of the sandy material plugging the drain. However, two drains were flushed and no appreciable amount of silt was found and the flows did not increase. Consequently the perforations were placed up, so the intercepted water would be carried out of the slide area.

Movement in this area has continued. The interception of the ground water was not considered great enough for the installation to be considered a successful one.

This installation is used to illustrate that horizontal drains are not always successful, even though preliminary investigation and study indicates that their installation would be an effective and economical means of slide control.

SUMMARY

It is the opinion of the writers, based on their experience and the experience of other personnel in the Materials and Research Department of the California Division of Highways that:

1. Installation of drains should be preceded by investigation and planning by competent engineers with thorough knowledge of soils and geology.

2. The actual installation should be made by trained personnel who are proficient in this type of work.

3. The field work should be under the supervision of personnel with sufficient engineering and geologic background and with the authority to make changes in the installation as the need arises based on available information.

4. Much progress has been made in the development and utilization of available equipment, but there is still a large field for development in this direction.

5. Systematic inspection and maintenance is a vital part of the overall program of an effective horizontal drain installation.

6. Horizontal drains have a definite place in the correction and prevention of slides in the design, construction and maintenance of embankments and cuts, and when properly planned, installed and maintained they are effective.

ACKNOWLEDGMENTS

The writers are indebted to the staff of the Materials and Research Department of the California Division of Highways under the direction of Mr. F. N. Hveem for their assistance and comments in the preparation of this paper, especially Mr. A. W. Root, Mr. Wesley Gray, Mr. Bruce Utt, and Mrs. Margaret Lark.

BIBLIOGRAPHY

1. T. E. Stanton - "California Experience in Stabilizing Earth Slopes Through the Installation of Horizontal Drains by the Hydrauger Method," Proceedings of the Second International Soil Conference on Soil Mechanics and Foundation Engineering - June 1948.
2. A. D. Hirsch - "Stabilizing Earth Slopes Through Installation of Drains with Hydrauger Equipment." Paper presented at the WASHO Conference at Reno, Nevada, June 13, 1950.
3. A. W. Root - "California Experience in Correction of Landslides and Slipouts." Paper presented at the ASCE San Francisco Convention, March 1953.
4. A. W. Root - "Horizontal Drill - New California Approach in Landslide Control," California Highways and Public Works, March-April 1955.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

A REVIEW OF THE THEORIES FOR SAND DRAINS

F. E. Richart, Jr.* A.M. ASCE
(Proc. Paper 1301)

SYNOPSIS

The existing theories for vertical consolidation of clays by vertical flow of water and by radial flow to a drain well have been studied and found to be satisfactory within the limits of the assumptions. A further study showed that the effect of considering void ratio as a variable did not significantly change the consolidation-time characteristics of vertical consolidation by vertical flow. Thus, including the effects of variable void ratio does not contribute toward the explanation of secondary consolidation.

From Barron's work on consolidation by flow toward drain wells, it was shown that a drain well having a smeared zone at its periphery can be considered as an equivalent "ideal" well of reduced diameter. Diagrams are included for quantitative evaluation of this relation. An example is included to show the effectiveness of even a small diameter ideal well in reducing the time for consolidation.

Numerical procedures were found to be versatile aids for solving the classical consolidation problems as well as for considering consolidation under a variety of conditions. Variable rates of loading, variable soil properties, layered systems, etc. can be readily included in the treatment of consolidation problems by these methods.

INTRODUCTION

The purpose of a drain well is to provide an easier path for the excess water to follow as it is squeezed out of a soil layer during consolidation. Thus, an effective well will accelerate the process of consolidation.

However, in practice such installations of drain wells, usually composed of sand columns and hence called "sand drains," have met with varied success. In order to study the causes of the successes or failures of various

Note: Discussion open until December 1, 1957. Paper 1301 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83, No. SM 3, July, 1957.

*Prof. of Civ. Eng., Univ. of Florida, Gainesville, Fla.

installations, it is instructive to review the factors involved in the performance of a drain well and, if possible, to evaluate the importance of each parameter. The following pages contain a review of the analytical approaches to the problem and a brief discussion of the effects of the more important variables involved.

Notation:

The letters and symbols used in the text are defined as they appear and are assembled in Appendix I.

Theory of Consolidation

Since the sand drain is merely an auxiliary device to expedite the process of consolidation, it is evident the consolidation of a soil layer is the fundamental subject to be considered. The theory of consolidation presented by K. Terzaghi(1,2,3) forms the basis of conventional procedures for predicting the time rate of thickness decrease of clay layers under load. The assumptions made in establishing the theory are:

1. The voids in a soil are completely filled with an imcompressible fluid, which is water.
2. The solid components of the soil are incompressible.
3. Darcy's law is valid.
4. The coefficient of permeability, k, is a constant.
5. The time lag of consolidation is due entirely to the low permeability of the soil.

Additional assumptions usually adopted unless it is specifically stated otherwise are that the soil is laterally confined, and that the coefficient of compressibility, a_v is a constant for the range of pressure considered. The assumption of lateral confinement restricts application of the theoretical solutions to conditions in which the lateral deformations in the consolidating material are small with respect to the vertical deformations. The assumptions for the consolidation theory are discussed more completely in reference 4, p. 266.

Utilizing the above assumptions, as well as the further assumptions that variations in the void ratio, e, are limited to small values so that (1 + e) may be treated as a constant, and the conditions of equilibrium of flow of water through an elemental volume of soil, Terzaghi established the differential equation of one-dimensional consolidation as

$$\frac{\partial u}{\partial t} = \frac{k(1+e)}{a_v \gamma_w} \frac{\partial^2 u}{\partial z^2} = c_v \frac{\partial^2 u}{\partial z^2} \qquad (1)$$

In Eq. (1), the excess pore water pressure, u, is expressed as a function of its vertical position in space, z, and time, t. The coefficient of consolidation, c_v is a constant since the unit weight of water, ∂_w, is a constant and the other terms k, a , and (1 + e) have been assumed to be constants.

Eq. (1) defines the vertical consolidation of a loaded clay layer due to a vertical flow of water. By a procedure similar to that used to derive Eq. (1),

the equations for vertical consolidation due to a two-dimensional and three-dimensional flow can be expressed in cartesian or cylindrical coordinates. When both radial and vertical flow of water exist so that the resultant flow path is inclined, the consolidation equation can be written in terms of cylindrical coordinates as

$$\frac{\partial u}{\partial t} = c_{v_r}\left\{\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r}\right\} + c_v \frac{\partial^2 u}{\partial z^2} \tag{2}$$

In Eq. (2) the coefficients of vertical consolidation due to radial flow (c_{v_r}) has been assumed to be different from the coefficient of vertical consolidation due to vertical flow of water (c_v).

Consolidation Equation Considering Void Ration as a Variable

If the change of void ratio, e, is appreciable such that it is no longer satisfactory to treat (1 + e) as a constant, a one-dimensional equation of consolidation may be derived taking this into account. The expression, derived in Appendix II, is

$$\frac{\partial u}{\partial t} = \frac{k}{a_v \gamma_w}\frac{1}{(1+e)}\frac{\partial^2 u}{\partial h^2} - \frac{k}{\gamma_w (1+e)^2}\left(\frac{\partial u}{\partial h}\right)^2 \tag{3}$$

In Eq. (3) e, u, t, and h, are variables while k, a_v, and γ_w are considered to be constants.

The distance increment, h, is based upon the equivalent height of the volume of solids in a given volume of soil. That is, for a soil element of unit area and height dz, the corresponding height of the volume of solids is dh. The relation between these distance elements, shown in Fig. 13 is,

$$(1+e)\,dh = dz \tag{4}$$

Consolidation by Vertical Flow of Water Only

For a horizontal layer of clay of thickness 2H, the top of the layer may be designated as the origin of the coordinate, z, which is measured positive downward.

The excess pore water pressure, u, which exists in this layer as a result of an applied vertical load is determined by Eq. (1) or Eq. (3). Since Eq. (1) is simpler, in that it defines u as a function of position, z, and time, t, only, it will be considered first. The manner in which u varies with z and t depends upon the boundary conditions.

For the condition of free drainage at the upper and lower boundaries of the clay layer (i.e. at z = 0 and z = 2H) the value of excess pore water pressure at these boundaries will be zero at any time. At the plane of symmetry, z = H, no water will flow in a vertical direction. And at an infinite time the

excess pore water pressure will be zero throughout the entire soil layer. Expressed in terms of symbols, these boundary conditions are, for

$$\left.\begin{array}{lll} 0 \leq t \leq \infty & \text{at } z = 0, \; z = 2H, & u = 0 \\ 0 \leq t \leq \infty & \text{at } z = H & \dfrac{\partial u}{\partial z} = 0 \\ t = \infty & \text{at } 0 \leq z \leq 2H, & u = 0 \end{array}\right\} \tag{5}$$

In addition, there is a boundary condition which is defined by the initial excess pressure throughout the layer, which is entirely carried by the pore water. For the case of equal initial excess pore water pressure throughout the soil layer it has the value u_o. This defines a further boundary condition for

$$t = 0 \quad \text{at} \quad 0 \leq z \leq 2H, \quad u = u_o \tag{6}$$

If the assumption is satisfied that the total settlement is small compared to the thickness of the clay layer, (i.e. $(1 + e) \approx$ const.) then utilizing the above boundary conditions, the solution of Eq. (1) may be obtained by means of a Fourier series as

$$u = \frac{4}{\pi} u_o \sum_{N=0}^{N=\infty} \frac{1}{2N+1} \sin\left[\frac{(2N+1)\pi z}{2H}\right] \mathcal{E}^{-(2N+1)^2 \frac{\pi^2}{4} T_v} \tag{7}$$

where

$$\left.\begin{array}{l} \mathcal{E} = 2.718 \cdots \text{ and} \\ T_v = \dfrac{c_v}{H^2} \cdot t \end{array}\right\} \tag{8}$$

T_v represents an independent dimensionless variable called the "time factor," which is customarily used as the abscissa along with the ratio $\frac{u}{u_o}$ as ordinate to describe graphically the consolidation-time relationship established by Eq. (7).

Solutions of Eq. (1) for a number of different boundary conditions are given in ref. 3.

Effect of Variable Void Ratio

An exact analytical solution for Eq. (3), satisfying the boundary conditions corresponding to Eqs. (5) and (6), would likely be quite complicated. However it was found possible to get an approximate solution for several

examples through the use of the difference equation procedure. Eq. (3) expressed in terms of finite differences in Appendix II and the method of solution is indicated.

Three examples were studied for which the initial void ratio, e_o, and the final void ratio, e_2, had the values of (a) $e_o = 0.65$, $e_2 = 0.55$, (b) $e_o = 1.00$, $e_2 = 0.80$, and (c) $e_o = 0.90$, $e_2 = 0.40$. For cases (a), (b), and (c) the final thicknesses of the clay layers were 94%, 90%, and 74%, respectively, of the original thicknesses of these layers.

The consolidation-time curves for the three cases considering void ratio as a variable are shown on Fig. 1 along with the curves obtained by the Terzaghi theory. Two curves are shown for the latter, one as obtained using Eq. (7) and the other obtained by difference equations in which the same space interval (0.25H) was used as for the solution of the variable void ratio problems.

The general shape of the curves including the variable void ratio is very similar to that of the curves obtained by the Terzaghi theory. The most significant effect of taking void ratio changes into consideration is exhibited by the position of these curves below and to the left of the usual curves. This indicates a slightly greater degree of consolidation at any particular value of time, as might be expected since the term $(1 + e)$ appears in the denominator of the right side of Eq. (3). Thus the Terzaghi theory gives a conservative estimate of the time required to reach a given degree of consolidation.

Since the difference in consolidation-time relationships by these two methods is unimportant when compared to the errors involved in establishing the soil constants, it is not necessary to introduce the added complication of variations in void ratio even for moderate changes in thickness of the clay layer.

The curves of Fig. 1 and the discussion included in Appendix II indicate that taking the variable void ratio into account does not contribute to the explanation of "secondary consolidation."

Simplification of Analysis for Consolidation by Two- or Three-Dimensional Flow

Consolidation by vertical flow alone involves only two variables, time, and depth z. For consolidation by two-dimensional flow, the variables are x, z, and time, while for three-dimensional flow they are x, y, z, and time. Thus the general solution for consolidation by three-dimensional flow, for a given set of boundary conditions, may become quite involved mathematically.

Fortunately, the method of separation of variables can be applied to this problem, as demonstrated by A.B. Newman[4] and later by N. Carrillo[5]. By use of this procedure, the expression for the resultant excess pore water pressure can be evaluated in terms of component solutions, which may be combined at the final step in the analysis. For example, for consolidation by two-dimensional flow the solution containing the variables x, z, and t can be determined by first evaluating u_x (which is a function of x and t), then u_z (which is a function of z and t), and combining these for a particular point in space at a particular time. The relation to be satisfied in the combining procedure is

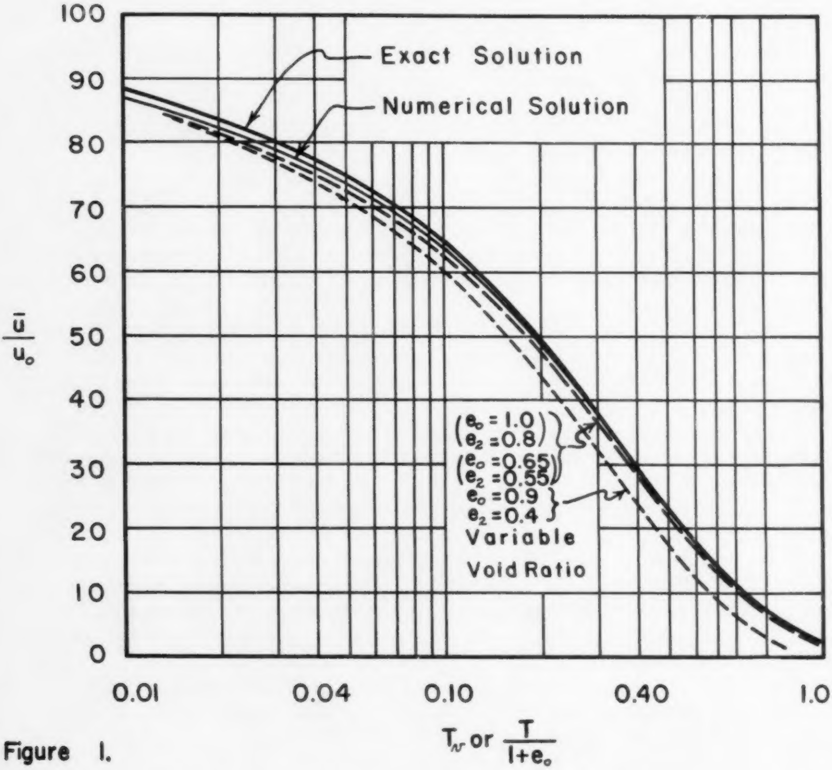


Figure 1.

Pore Water Pressure vs. Time Curves for Vertical Water Flow, Including Effects of Variable Void Ratio

$$\frac{u}{u_o} = \frac{u_x}{u_o} \cdot \frac{u_z}{u_o}, \text{ or } u = \frac{u_x u_z}{u_o} \tag{9}$$

A similar relation holds regarding the average values of pore water pressure, as

$$\bar{u} = \frac{\bar{u}_x \bar{u}_z}{\bar{u}_o} \tag{10}$$

Eq. (10) is the more convenient relationship and can be used for homogeneous layers.

Vertical Consolidation Due to Radial Flow of Water

The treatment of consolidation due to radial flow is an extension of the Terzaghi consolidation theory. A solution was presented in 1935 by L. Rendulic,(6) working under Terzaghi's direction, and more comprehensive study of the influence of ideal wells on consolidation was made by R. A. Barron(7) during 1940-1942.

The first generally available design information on this topic was given by Terzaghi in his book "Theoretical Soil Mechanics,(3) which was later supplemented by a paper which he published in 1945.(8) In 1948, Barron(9) presented a complete summary of the theory of sand drains and he also included new theories which took into account deviations from the ideal well conditions.

The two papers by Barron(7,9) constitute the principal analytical studies available at present of radial flow toward a drain well and the resulting consolidation of the clay. He considered two types of vertical strain which might occur in the clay layer, (a) involved the condition of "free vertical strain" which would result from a uniform distribution of surface load, while (b) involved "equal vertical strains" which would result from imposing the same vertical deformation at all points on the surface. For both strain conditions he included an analysis of the effect of "smear" of the soil near the well boundary, and the effect of resistance to flow through the well itself.

"Smear" is the term used to define the wiping action provided by the casing or hollow mandrel used to form the well, as it is driven down into the soil, and then pulled out after it has been filled with sand. This action tends to smear the soil at the well periphery. For a soil with originally a much greater permeability in the horizontal than in the vertical direction, the smeared zone which has been created forms a barrier to the horizontal flow of water, thereby slowing down considerably the process of consolidation.

"Free Strain" Consolidation with No Smear and No Well Resistance

A regular pattern of vertical drain wells, as shown on Fig. 2, will permit a radial as well as a vertical flow of water from the clay layer if it is subjected to an increase in pressure. Since in a previous section it was shown that the effects due to vertical flow and due to radial flow can be evaluated separately, then be combined to give the final consolidation behavior, the radial flow consolidation only is considered here.

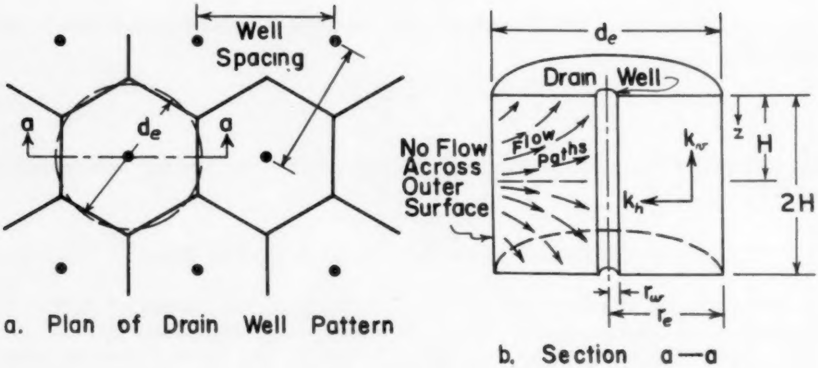


Figure 2. Plan of Drain Wells and Concept of Flow within Zone of Influence of each Well.

As indicated on Fig. 2, for a triangular spacing of drain wells, a zone of influence exists which has a hexagonal plan form. By approximating the hexagon by a circle of equivalent diameter, d_e, this can be used as the outer limit of the zone of influence of each drain well. Thus it becomes sufficient to consider the radial flow and resulting consolidation of a volume of soil of unit thickness which is contained between the distances d_e (diameter of well influence) and d_w (diameter of the drain well).

By eliminating the consideration of vertical flow, Eq. (2) becomes

$$\frac{\partial u}{\partial t} = c_{v_r}\left\{\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r}\right\} \tag{11}$$

which is the equation for consolidation expressed in terms of radial coordinates. The boundary conditions which must be satisfied are:

(a) The initial pore water pressure, u_o, is uniform throughout the soil mass when $t = 0$.

(b) The excess pore water pressure at the drain well surface (r_w) is zero when $t>0$.

(c) The external radius, r_e, is considered impervious because of symmetry. Thus, $\frac{\partial u}{\partial r} = 0$ when $r = r_e$

The solution of Eq. (11), subject to the boundary conditions indicated above, leads to the following expressions for u_r (the pore water pressure at any location, r, at any time, t, which exists as a result of radial flow only) and $\bar{u}_r$ (the average value of u_r throughout the soil mass at any time, t):

$$u_r = u_o \sum_{\alpha_1, \alpha_2, \alpha_3 \cdots}^{\alpha=\infty} \frac{-2\, U_1(\alpha)\; U_0\left(\frac{\alpha r}{r_w}\right)}{\alpha\left\{n^2 U_0^2(\alpha n) - U_1^2(\alpha)\right\}}\, \varepsilon^{-4\alpha^2 n^2 T_h} \tag{12}$$

and

$$\bar{u}_r = u_o \sum_{\alpha_1, \alpha_2, \alpha_3 \cdots}^{\alpha=\infty} \frac{4\, U_1^2(\alpha)}{\alpha^2(n^2-1)\left\{n^2 U_0^2(\alpha n) - U_1^2(\alpha)\right\}}\, \varepsilon^{-4\alpha^2 n^2 T_h} \tag{13}$$

where

$$\left.\begin{aligned} U_1(\alpha) &= J_1(\alpha)\, Y_0(\alpha) - Y_1(\alpha)\, J_0(\alpha) \\ U_0(\alpha n) &= J_0(\alpha n)\, Y_0(\alpha) - Y_0(\alpha n)\, J_0(\alpha) \\ U_0\left(\frac{\alpha r}{r_w}\right) &= J_0\left(\frac{\alpha r}{r_w}\right) Y_0(\alpha) - Y_0\left(\frac{\alpha r}{r_w}\right) J_0(\alpha) \end{aligned}\right\} \tag{14}$$

The important symbols used in Eqs. (12) and (13) are defined below:

J_o, J_1 = Bessel functions of the first kind, of zero order and first order, respectively.

Y_o, Y_1 = Bessel functions of the second kind, of zero order and first order, respectively.

$\alpha_1, \alpha_2, \alpha_3$, --- = roots of the Bessel functions which satisfy

$$J_1(\alpha n)\, Y_o(\alpha) - Y_1(\alpha n)\, J_o(\alpha) = 0.$$

n = ratio equal to $\frac{r_e}{r_w} = \frac{d_e}{d_w}$

T_h = time factor for consolidation by radial flow $= \frac{k_h(1+e)\,t}{a_v \gamma_w d_e^2}$

Solutions similar to Eqs. (12 and (13) were derived by Glover[10] for the analogous heat flow problem.

"Equal Strain" Consolidation with No Smear and No Well Resistance:

Under the condition of "free strain" it was implied that the settlements at the surface did not change the distribution of the load to the soil. However, in an actual installation, the fact that consolidation proceeds faster near the drain well, thereby causing a greater surface settlement in that region, could very well cause a redistribution of the surface loading. This would be especially true if the loading material had any tendency to "arch" across such depressions. As an extreme case the arching action could redistribute the loads to the surface in such a fashion that the surface settlement is the same at all points. This is the condition of "equal vertical strains" for which Barron has also developed an analytical solution,

$$u_r = \frac{4\bar{u}}{d_e^2 F(n)} \left[r_e^2 \log_\varepsilon\left(\frac{r}{r_w}\right) - \frac{r^2 - r_w^2}{2} \right] \tag{15}$$

in which

$$\bar{u} = u_o \varepsilon^{\lambda} \tag{16}$$

$$\lambda = \frac{-8T_h}{F(n)} \tag{17}$$

and

$$F(n) = \frac{n^2}{n^2-1} \log_\varepsilon(n) - \frac{3n^2-1}{4n^2} \tag{18}$$

The initial distribution of hydrostatic excess pressure is not uniform, but may be computed from Eq. (15) for $T_h = \lambda = 0$. Curves showing the relation between the average pore water pressure $\bar{u}$ and the time factor, T_h, can also

be obtained from Eq. (16). Such surves for n = 5, 10, 40, and 100 are shown on Fig. 3, along with the corresponding curves determined by the "free strain" case.

Comparison of "Free Strain" and Equal Strain Solutions

The difference between the results obtained by the two rather extreme considerations of the process of consolidation is small, particularly for the curves representing values of n greater than about 10. For n = 5 the discrepancy is somewhat greater for the first part of consolidation, but above about 50% consolidation, the curves are almost identical.

Since the results are nearly identical, but the time needed to evaluate Eq. (13) is of the order of ten to fifteen times that needed to evaluate Eq. (16), the equal strain solution is preferable. Both Barron(9) and Kjellman(11) recommended use of the equal strain solution.

From Fig. 3 it should be noted that the curves representing the equal strain solutions for different values of n have the same shape, but are displaced horizontally. The location of the consolidation-time curve for any particular value of n depends upon the value of λ as determined from Eq. (17).

Effect of Peripheral Smear

The remolded or smeared zone at the periphery of the drain well creates an additional resistance which must be overcome by the excess water which is being expelled. This additional resistance retards the consolidation process.

The smeared zone will not be uniform or homogeneous with regard to the soil properties. Very likely it consists of a thin layer of actual smear plus an adjacent region in which the soil has undergone a considerable amount of disturbance. The amount of disturbance decreases with distance away from the well periphery.

However, in order to include the effects of smear and remolding, Barron(7) has considered that the smeared zone contains a homogeneous material which has soil properties different from those in the remaining material in the soil cylinder. The important quantities to be considered in analyzing the effects of this smeared region are (a) the ratio, s, of the radius of the smeared zone to the well radius ($s = \frac{r_s}{r_w}$) and (b) the ratio of the coefficients of horizontal permeability in the undisturbed soil (k_h) and in the smeared zone (k_s). For s = 1, there is no thickness to the smeared ring, and if $\frac{k_h}{k_s} = 1$, then the disturbed zone does not change the water flow characteristics of the soil cylinder.

He also assumed that the smeared zone will consolidate very fast, thus its consolidation can be ignored and the zone can be treated as an incompressible material.

By ignoring the consolidation of the smeared zone, he was able to treat this region as one in which flow exists between one boundary value which is zero and another boundary value which is time dependent. This is a very reasonable approximation to the actual behavior, since the excess pore

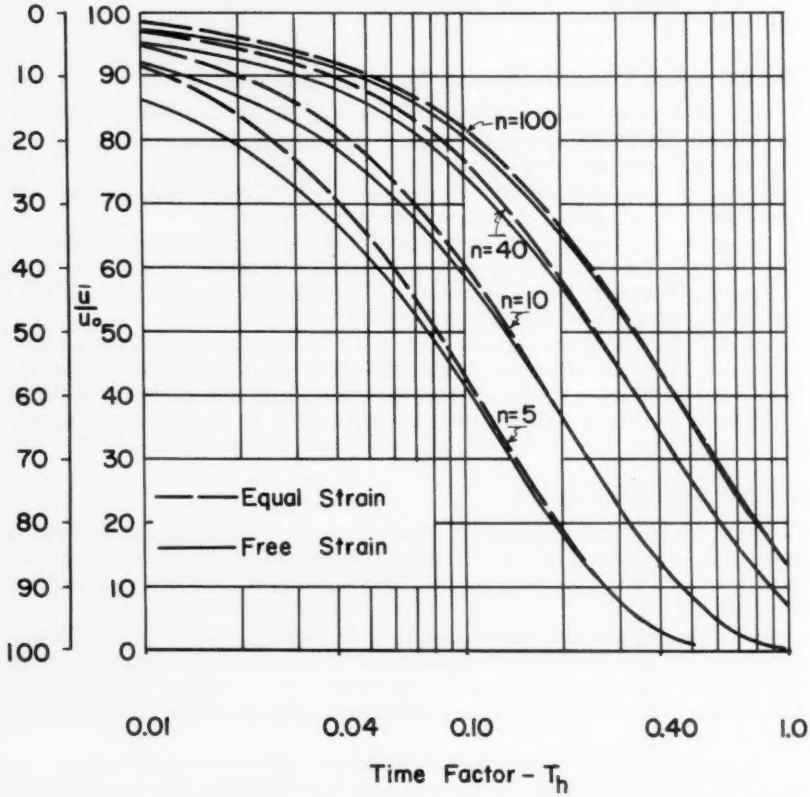


Figure 3. Consolidation vs. Time Curves for Cases of "Free Strain" and "Equal Strain"

water pressure within the smeared region would quickly dissipate into this steady flow condition. For an extreme case where n = 5, s = 2, and $\frac{k_h}{k_s} = 2$, and consolidation of the smeared zone was considered, the steady flow condition was reached at about $T_h = 0.025$.

Equal Strain with Smear

Barron's solution for the excess pore water pressure in a soil cylinder undergoing equal vertical strains and containing a smeared region around the drain well is

$$u_r = \bar{u}_r \frac{\left[\log_\varepsilon\left(\frac{r}{r_s}\right) - \frac{r^2 - r_s^2}{2r_e^2} + \frac{k_h}{k_s}\left(\frac{n^2 - s^2}{n^2}\right)\log_\varepsilon(s)\right]}{\nu} \tag{19}$$

in which

$$\nu = \left[\frac{n^2}{n^2 - s^2}\log_\varepsilon\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{s^2}{4n^2} + \frac{k_h}{k_s}\left(\frac{n^2 - s^2}{n^2}\right)\log_\varepsilon(s)\right] \tag{20}$$

and $\bar{u}_r$ can be determined from

$$\bar{u}_r = u_o \varepsilon^{\xi} \tag{21}$$

in which

$$\xi = \frac{-8\,T_h}{\nu} \tag{22}$$

Evaluation of Smear Effects as Equivalent Changes of Well Diameter

By comparing the equations it is seen that Eqs. (19) and (15), (20) and (18), (21) and (16), and (22) and (17) become identical when s = 1. Also, Eqs. (16) and (21), and (17) and (22) are similar.

For the ideal wells, the position of the consolidations-time curve for any particular value of n depends on the value of F(n) (Eq. 18) which is a function of n only. For the wells with smear, the position of the resulting consolidation-time curves depends upon ν (Eq. 20) which is a function of n, s, and $\frac{k_h}{k_s}$

Thus, it is possible to interpret various combinations of n, s, and $\frac{k_h}{k_s}$ in the treatment of a well with smear as if they define an ideal well having a larger value of n. For a given radius of influence, a larger value of n determines the radius of an equivalent ideal well which is smaller than the

radius of the actual well which is surrounded by a zone of smear. The effect on consolidation of the soil cylinder caused by the introduction of a smeared zone at the well periphery is identical to the effect caused by reducing the size of the ideal well.

Fig. 4 shows the combinations of n, s, and $\frac{k_h}{k_s}$ which can be used to give the same value of ν. This diagram, obtained by evaluating Eq. (20), uses the value of ν as ordinate and the ratio $\frac{k_h}{k_s}$ as abscissa. Families of lines for each selected value of n show the influence of variations in s. Only the lines for $n = 5$ and $n = 15$ are shown on Fig. 4. However, by the use of Fig. 5, diagrams similar to Fig. 4 can be established for a wide range of values of n.

As an example of the use of Fig. 4, consider a drain well for which $n = 5$, $s = 1.2$, and $\frac{k_h}{k_s} = 7$. The value of ν determined from the diagram is 1.97. This corresponds to $F(n) = 1.97$ for a well having no smear $(s = 1)$, or it determines $n = 15$ for the equivalent ideal well. Evaluating this in numbers indicates that if the original diameter of well influence was 15 ft., the diameter of the drain well as 3 ft. (from $n = 5$), and the outer diameter of the smeared region was 3.6 ft. $(s = 1.2)$. The effect of the smeared region is to reduce the consolidation-time behavior to one identical to that of a 1 ft. diameter well $(n_{eff.} = 15)$ for which no smear is present. (See Fig. 6)

Effect of Well Resistance

The foregoing analyses have considered that there is unrestricted flow of the water through the drain well. Actually, head losses will occur due to the resistance of the well backfill material to flow. The magnitude of the head losses will depend upon the rate of flow, the size of the well, and the permeability of the material filling the well.

Barron has developed a solution for the case of equal vertical strain, with or without smear, for a material in which no vertical flow exists due to lack of permeability in the vertical direction, but $\frac{\partial u}{\partial z} \neq 0$.

However, for practical drain well installations for which n is about 7 to 15 and for $\frac{d_e}{H} \leq 1.0$, the effect on the consolidation behavior due to resistance of the drain wells should not be significant.

Example Considering Ideal Wells

In order to illustrate how some of this information can be of value in selecting the size and spacing of drain wells for a particular installation, the following values were chosen:

$a_v = 13 \times 10^{-5}$ cm/gm $\qquad k_v = 5 \times 10^{-8}$ cm/sec

$e = 1.50$ $\qquad k_h = 25 \times 10^{-8}$ cm/sec

$H = 10$ ft. $\qquad \Delta e = 0.125$

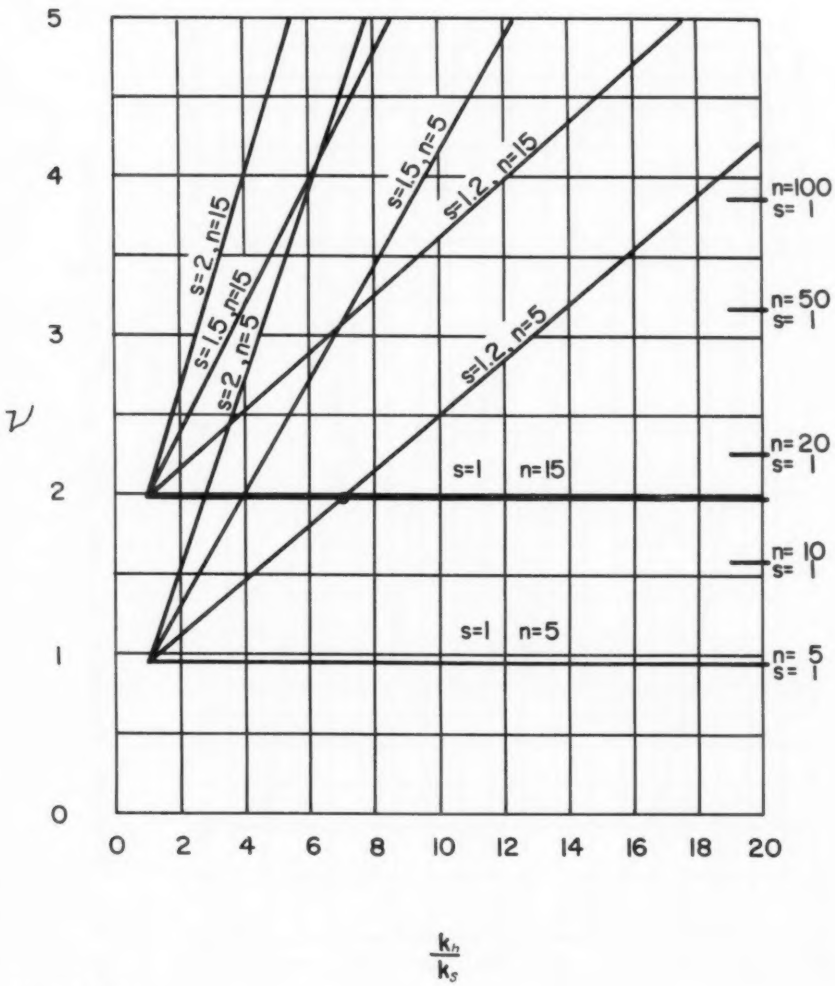


Figure 4.

Relations between $\frac{k_h}{k_s}$, ν, and s, for n=5 & n=15

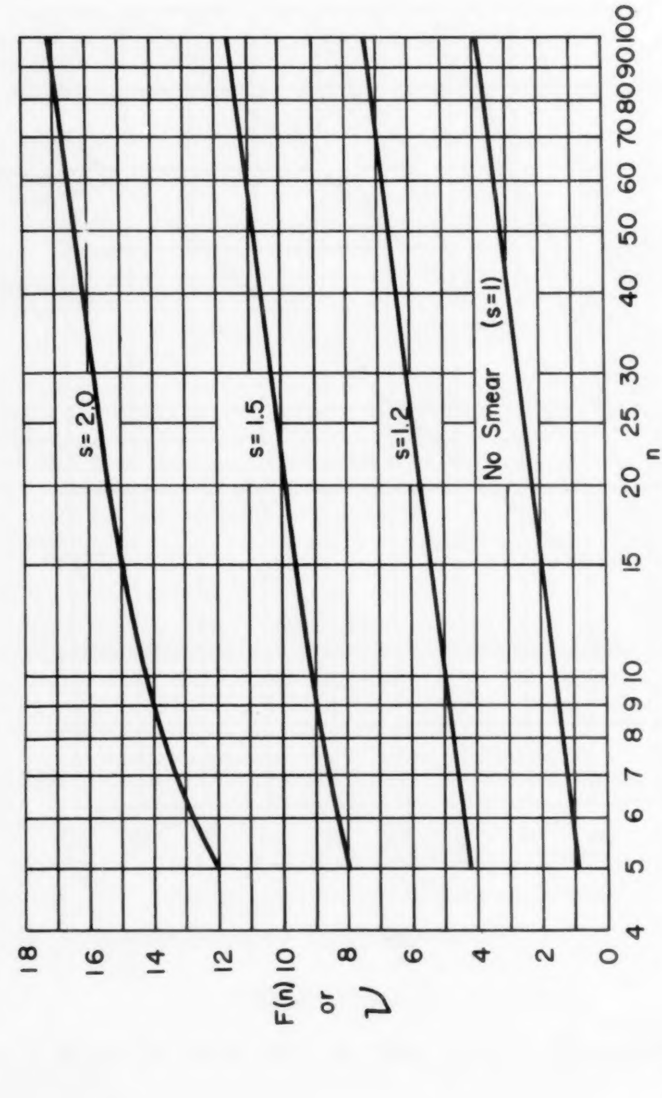
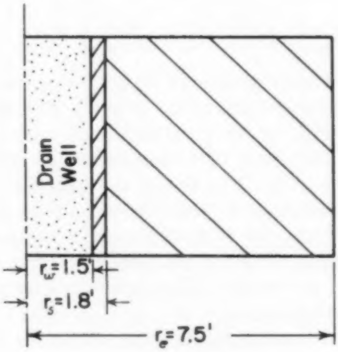


Figure 5. Relation between n and F(n) or ν for $\frac{k_h}{k_s} = 20$



(a.) Actual Well Installation
$n = 5$, $s = 1.2$, $\frac{k_h}{k_s} = 7$

(b.) Effective Well Installation
$n_{eff} = 15$

Figure 6. Actual and Equivalent Well Installations, both Corresponding to $\nu = 1.97$ on Figure 4.

These values determine $c_v = \frac{9.62}{10^4}$ cm^2/sec, which is of the same order of magnitude as the test results given by Terzaghi and Peck.[12] The clay layer is 20 ft. thick if drainage occurs at both top and bottom surfaces, or is 10 ft. thick if drainage occurs at the top surface only. The horizontal permeability is five times that in the vertical direction.

For the initial conditions, assume that there is no smeared zone and no well resistance, so that the consolidation behavior depends only upon the size and spacing of the drain wells.

Constant Well Diameter

To compare the effects of well spacing a well diameter of 12" was chosen and the percent consolidation vs. time curves are shown on Fig. 7. Fig. 7(a) shows the consolidation-time curves for radial or vertical drainage only, for three diameters of well influence (d_e = 5', 10', and 20'), and for three thicknesses of clay layer (H = 10', 20', and 30'). Fig. 7(b) shows the effect of a one foot diameter well with a diameter of influence of ten feet when it is introduced into each of the clay layers. The time for consolidation is reduced to a fraction of the time for vertical drainage. The use of closer spacing of drain wells would further reduce this time, as would be demonstrated by using the curve for $d_e = 5$ in Fig. 7(b) instead of that for $d_e = 10$.

It should be noted from Fig. 7(b), that the consolidation-time behavior due to combined radial and vertical drainage is nearly identical to that due to radial drainage alone, for $d_e = 10'_V$. This is influenced to a considerable extent by the conditions of $k_h = 5k_v$. If $k_h = k_v$, it would be necessary to consider a smaller well spacing, or smaller ratio $\frac{d_e}{H}$, for the radial flow behavior to dominate the consolidation-time behavior of the clay layer due to combined vertical and radial flow.

Constant Well Spacing

For the conditions involving a constant spacing of the wells, but allowing the diameter of the wells to vary, the consolidation vs. time curves are as shown on Fig. 8. The chosen well spacing was 10' which is the same as the thickness of the clay layer.

Fig. 8(a) shows the consolidation vs. time curves for the radial or vertical flow when they act independently. Fig. 8(b) shows the effect on consolidation due to combined flow, and for 90 percent consolidation the time amounts to

(a) 6.0 months (19.2% of time for vertical only) for d_w = 1.2"
(b) 3.9 months (12.5% of time for vertical only) for d_w = 6"
(c) 1.7 months (5.4% of time for vertical only) for d_w = 24"

Thus, even the 1.2 in. diameter well in a 10 ft. soil cylinder cuts the consolidation time to about one-fifth of the time required for 90 percent consolidation by vertical drainage only.

It is of particular importance to note that doubling of the diameter of well influence (d_e), or essentially the well spacing, causes an increase in the time for 90 percent consolidation by roughly a factor of 6 (Fig. 7a). On Fig. 8(a) it is seen that by reducing the diameter of drain well by a factor of 20 only increases the time for 90 percent consolidation by a factor of about 4.

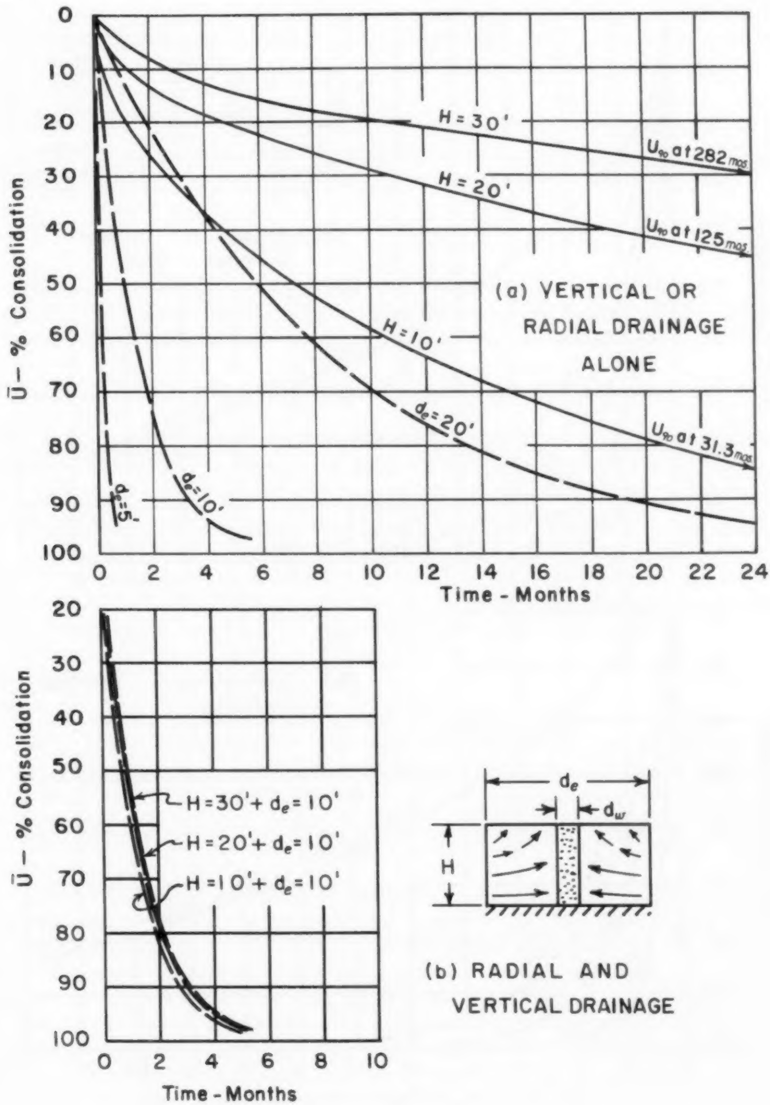


Figure 7. Effect of Drain Well on Time for Consolidation—for Various Thicknesses of Clay Layer

$$\left(d_w = 12'', \quad \frac{k_h}{k_v} = 5, \quad c_v = \frac{9.6}{10^4}\frac{cm^2}{sec}\right)$$

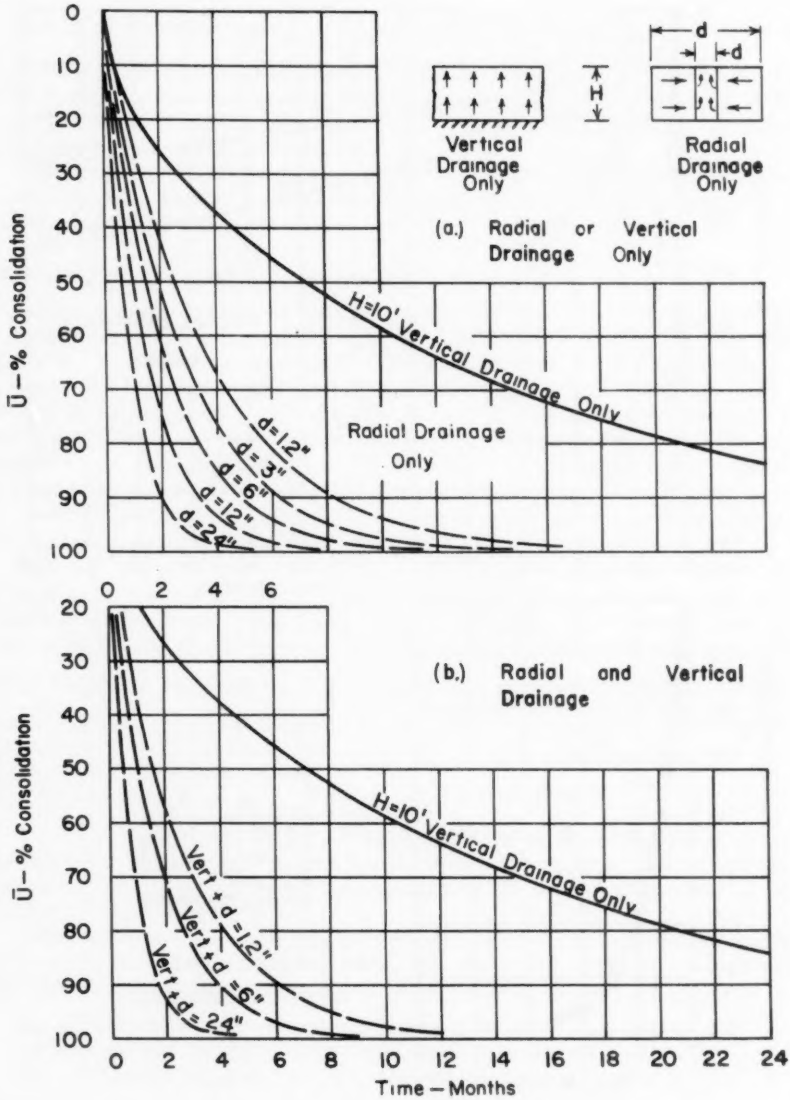


Figure 8. Effect of Size of Drain Well on Time for Consolidation $\left(H = 10', \quad d_e = 10', \quad \frac{k_h}{k_v} = 5, \quad c_v = \frac{9.6}{10^4} \frac{cm^2}{sec}\right)$

Consequently, the effectiveness of a given drain well installation is considerably more dependent upon the choice of well spacing than it is upon the well diameter.

Effect of Horizontal Permeability

In order to convert the non-dimensional time factor T_h into terms of t_h in months, it is necessary to use the equation,

$$t_h = \frac{T_h \gamma_w a_v d_e^2}{k_h (1+e)} \tag{23}$$

From Eq. (23) it is evident that the time for any given degree of consolidation is inversely proportional to the coefficient of permeability in the horizontal direction.

To illustrate the effect of $\frac{k_h}{k_v}$ on the time required to obtain 90 percent consolidation by combined vertical and radial flow, Fig. 9 has been prepared using two well systems (a) $d_e = 10'$, $d_w = 1.2''$, $n = 100$, and (b) $d_e = 10'$, $d_w = 12''$, and $n = 10$. Fig. 9 shows the very pronounced effect of the ratio $\frac{k_h}{k_v}$ for the two well systems, and even for the case of $\frac{k_h}{k_v} = 1.0$, the time for 90 percent consolidation is reduced to less than 55 percent of the time required when no wells are present. The 55 percent time corresponds to the well system made up of 1.2 in. diameter wells spaced every 10 ft.

Discussion of Example

Of course, this 1.2 in. diameter well considered above is an "ideal well" for which there is no smear and no well resistance. But the significant point is that such a small well can be so effective. If the well spacing was reduced to three or four feet, the effectiveness of such a 1.2 in. diameter well would be increased many times. Thus, the basic idea of the cardboard "wicks" spaced at about 4 ft., as described by W. Kjellman[11] is based upon sound theoretical considerations.

In order to design a system of drain wells, some numerical information is required about the well resistance, the extent or radius of the smear zone, and the permeability of the material in the smear zone. Then using the desired diameter of the ideal well, for a given well spacing, Figs. 4 or 5 can be used to determine the actual size of well needed. That is, the actual size of well to be constructed is reduced in effectiveness by the smear and well resistance until it behaves similar to an ideal well of smaller diameter. Then the behavior of the equivalent ideal well may be studied thoroughly with the aid of the many diagrams already prepared by Barron.[7,9]

Solution of the Consolidation Problem by Numerical Methods

Since the "exact" solutions for the consolidation problem become unwieldy for rather modest departures from the simplified case, as when "smear" is introduced into the problem of consolidation by radial flow, it is desirable to investigate methods which lead to approximate results. In recent years, an

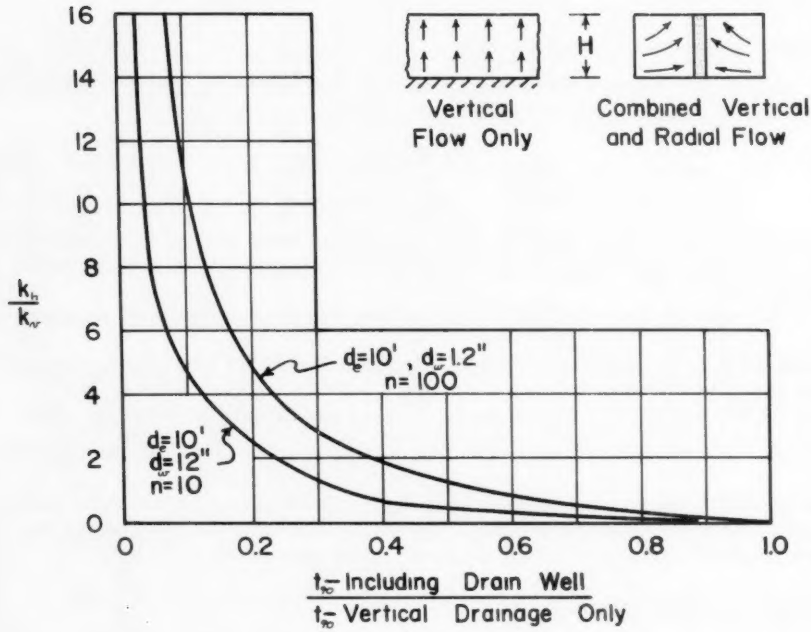


Figure 9. Influence of Permeability Ratio on Time for 90% Consolidation $\left(c_v = \frac{9.6}{10^4} \frac{cm^2}{sec}\right)$

appreciable amount of work has been directed toward application of the difference equation procedure to the problem of transient flow of heat in solids. (For example[13, 14, 15, 16]). The results of these studies may be used directly in analogous problems such as consolidation of clays.

Explicit Expressions

One procedure which has been used extensively for numerical solutions of both the heat flow and consolidation[17, 18] equations permits a solution for the value of the dependent variable at a given point in space and time by considering only values of this variable at points in space at a previous time. The solution thus amounts to a step-by-step determination of these values at points in space as they vary along the time coordinate.

The expression resulting from replacing Eq. (1), for example, by its differences equivalent, is

$$\frac{u_{0,t+\Delta t} - u_{0,t}}{\Delta t} = c_v \left\{ \frac{u_{2,t} - 2u_{0,t} + u_{4,t}}{(\Delta z)^2} \right\} \tag{24}$$

In Eq. (24) the first subscript for the "u" terms denotes the position in space of the point under consideration (also shown on Fig. 11) and the second subscript denotes the time, with Δt equal to the time interval.

Eq. (24) is usually rearranged, for convenience in computation, as

$$u_{0,t+\Delta t} = u_{0,t} + A\left\{ u_{2,t} - 2u_{0,t} + u_{4,t} \right\} \tag{25}$$

in which the constant A represents,

$$A = \frac{c_v \Delta t}{(\Delta z)^2} \quad \text{or,} \quad A = \frac{\Delta T}{(\Delta \zeta)^2} \tag{26}$$

if ζ is defined as

$$\zeta = \frac{z}{H}, \quad \text{and} \quad \Delta z = H(\Delta \zeta) \tag{27}$$

Previous studies[16] of the heat flow equation have determined that if $A > 0.5$, the values of u determined by Eq. (25) diverge, and for $A \leq 0.5$, they converge. Also Eq. (26) establishes a definite relation between the increments of time and space for a particular choice of A. Thus, even for a rather crude space network, such as $\Delta \zeta = 1/4$, the maximum allowable time increment is $\Delta T_V = 0.03125$, with the result that numerous steps in time are required in the solution. Since this is a step-by-step procedure, in order to determine the excess pore water distribution at a particular time, it is necessary to work up to that time by using equal sized time intervals, starting from $T_V = t = 0$.

Implicit Expressions

The size of the time interval dictated by the stability consideration of the explicit scheme is often inconveniently small, therefore a method permitting

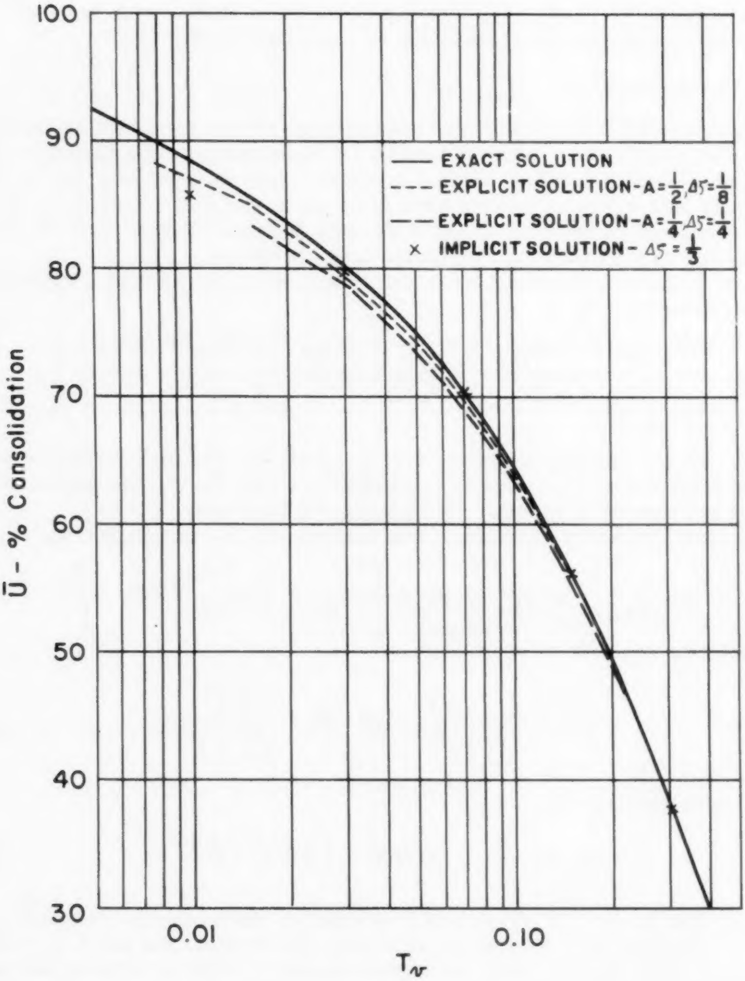
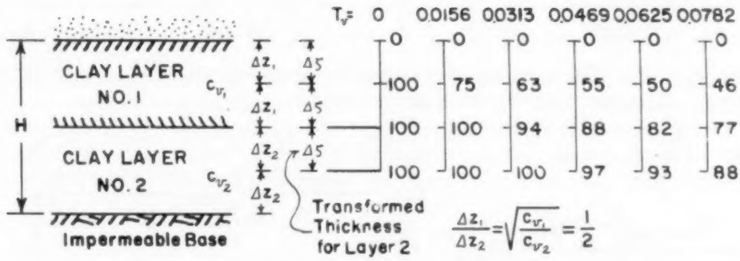
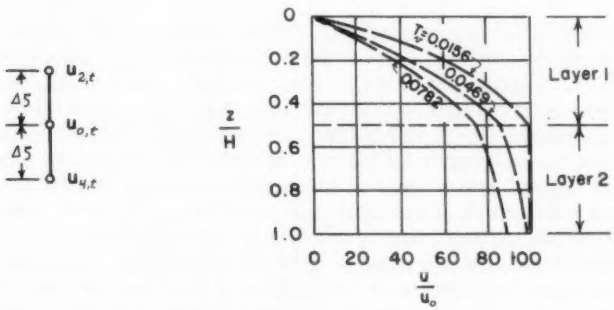


Figure 10. Consolidation vs. Time Curves for Vertical Flow Computed by Exact and Numerical Methods



(a.) Partial Solution for Variations in $\frac{u}{u_0}$ with Time and Depth for Layered System



(b.) Isochrones for Layered System

Figure 11. Numerical Procedures for Consolidation of a Two-Layer System due to Vertical Flow

larger time increments is desirable. This has been accomplished by "turning around" the difference equation, resulting in the form, (corresponding to Eq. (1))

$$\frac{u_{o,t+\Delta t} - u_{o,t}}{\Delta t} = c_v \left\{ \frac{u_{2,t+\Delta t} - 2u_{o,t+\Delta t} + u_{4,t+\Delta t}}{(\Delta z)^2} \right\} \tag{28}$$

The form of Eq. (28) used for computation[16] considers the average value of the space relations over the time interval as,

$$\frac{u_{o,t+\Delta t} - u_{o,t}}{\Delta t} = \frac{c_v}{2} \left\{ \begin{aligned} & \frac{u_{2,t+\Delta t} - 2u_{o,t+\Delta t} + u_{4,t+\Delta t}}{(\Delta z)^2} \\ & + \frac{u_{2,t} - 2u_{o,t} + u_{4,t}}{(\Delta z)^2} \end{aligned} \right\} \tag{29}$$

Eq. (28) is an implicit relation for the values of excess pore water pressure at the time $t + \Delta t$; that is, these unknowns occur simultaneously in the equations. This requires the solution of a set of simultaneous equations for each time interval adopted, but the size of the time interval is arbitrary and may be changed during the solution of a problem. The formulas are stable and place no restriction on the size of the space or time increments. However, if a small space interval is required in order to gain accuracy, this increases the number of simultaneous equations. For such cases, the use of digital computing machinery is nearly always required.

Although the implicit scheme does not place restrictions on the size of the time or space increments as a criterion of stability, the fact that the finite differences must be reasonable approximations to the corresponding differentials does prohibit large increments, particularly in time. The size of the allowable time increment depends on the rate of change of pore water pressure within that increment. Criteria for this and for related procedures have been given by Tung and Newmark[19].

Numerical Solutions of Vertical Consolidation by Vertical Flow

Eq. (24) was evaluated for two cases, $A = 1/4$, $\zeta = 1/4$ and $A = 1/2$, $\zeta = 1/8$, for which the resulting time intervals were $\Delta T = 0.015625$ and 0.0078125, respectively. The curves thus obtained are shown on Fig. 10 as well as the curve corresponding to the exact solution and the agreement is good, particularly for values of $T_v > 0.10$.

Eq. (29) was also evaluated for comparison. Using $\zeta = 1/3$ and time increments which doubled in size at each time step, starting with $T_v = 0.010$ (i.e., $\Delta T_{v1} = 0.010$, $\Delta T_{v2} = 0.020$, $\Delta T_{v3} = 0.040$, etc) a curve was obtained which agreed more closely with the exact solution than did those computed by the explicit method.

Since the explicit method permits the use of a fine space mesh with little increase in computational effort above that for a coarse mesh, it appears desirable to use this scheme for small values of time for which u is changing rapidly. Then for larger values of time, it would be convenient to switch to a coarse time mesh and use the implicit scheme.

Vertical Consolidation by Vertical Flow for Layered Systems

A clay stratum, made up of two or more horizontal layers, may also be treated by numerical procedures. At the interface between any two layers the conditions of equilibrium require that the velocity of flow leaving one layer must be equal to the velocity of flow entering the other. Thus for two materials having coefficients of permeability of k_{v_1}, and k_{v_2}, this condition at the interface requires that

$$k_{v_1}\left(\frac{\partial u}{\partial z_1}\right) = k_{v_2}\left(\frac{\partial u}{\partial z_2}\right) \tag{30}$$

The two layers also have different coefficients of consolidation, c_v, which are dependent upon a_v as well as k_v. In order to simplify the numerical procedure, it is convenient to adjust the size of the space intervals so that the term A remains the same in both layers for the same Δt. Thus,

$$A = \frac{c_{v_1}\Delta t}{(\Delta z_1)^2} = \frac{c_{v_2}\Delta t}{(\Delta z_2)^2} \tag{31}$$

or

$$\frac{\Delta z_1}{\Delta z_2} = \sqrt{\frac{c_{v_1}}{c_{v_2}}} \tag{32}$$

Fig. 11 illustrates the numerical solution for a two-layered system, consisting of a top layer, 1, resting on a second layer, 2, of equal thickness. For the two layers, $c_{v_2} = 4c_{v_1}$, which determines for a choice of $\Delta\zeta = 0.25$ that the space increments are 0.25H in layer 1 and 0.50H in layer 2. In Fig. 11(c), the isochrones show a definite change in slope at the interface, which should correspond to Eq. (30).

Numerical Solution of Consolidation by Radial Flow

By expressing Eq. (11) in terms of finite differences we have

$$\frac{u_{0,t+\Delta t} - u_{0,t}}{\Delta t} = \frac{c_v}{\Delta r^2}\left[\left(1+\frac{\Delta r}{2r_0}\right)u_{1,t} + \left(1-\frac{\Delta r}{2r_0}\right)u_{3,t} - 2u_{0,t}\right] \tag{33}$$

or

$$u_{o,t+\Delta t} = \frac{c_r \Delta t}{\Delta r^2}\left[\left(1+\frac{\Delta r}{2r_o}\right)u_{1,t} + \left(1-\frac{\Delta r}{2r_o}\right)u_{3,t} - 2u_{o,t}\right] + u_{o,t} \tag{34}$$

The network points used in Eq. (34) are spaced to the right and to the left of the point o, for which the value of the excess pore water pressure is being determined.

From Eq. (34) it is evident that a different equation must be used for each point under consideration, due to the specific value of r at each location. This slows the computations a bit but does not complicate the solution otherwise. Fig. 12 illustrates the procedure for solution of a problem for radial flow in a homogeneous soil cylinder for the case of $n = 5$. By taking the average value of the excess pressure at any time, a curve of $\bar{u}_r$ vs. T_h can be constructed which corresponds to within a few percent of the values obtained by Barron's procedure.

Additional Variables Which can be Treated by the Numerical Procedure

Since the numerical procedure involves the building up of a solution throughout a series of time intervals, it is ideally suited for consideration of quantities which vary with time. For cases where the surface load is varied during consolidation, this can be introduced into the problem simply as an overall increase (or decrease) of the excess pore water pressure at a particular time. A gradually increasing load can be approximated by using a stairstep variation of load with time.

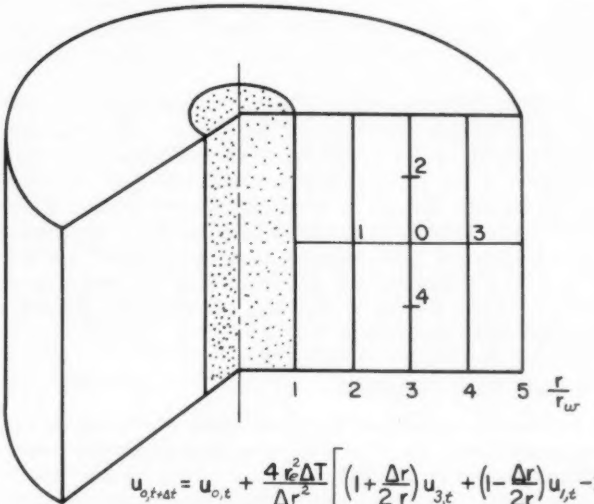
Variations of fundamental soil properties, such as the coefficients of consolidation or swelling, can be included as they vary both with time and space. That is, the intergranular pressure changes affect the soil properties, which may be adjusted accordingly during the period of consolidation. The soil properties are considered to be constant during an interval of time, Δt, in accordance with the assumptions of the theory of consolidation, but the value of these constants may be changed in successive time intervals. The soil constants may also have different initial values at the various space locations of the network points under consideration.

The flexibility of the numerical procedures also permits studies of two and three-dimensional consolidation to be made without excessive effort. For example, Gibson and Lumb[17] have obtained approximate solutions for the time rate of settlement of circular pervious and impervious footings, resulting from consolidation of the underlying clay.

Discussion of Use of Theories for Design or Analysis of Sand Drain Installations

Limitations

The theories for consolidation of clay layers including the effects of sand drains can be expected to give reasonable results only when the clay is comparable to the ideal material assumed as a basis for the theory. This



$$u_{o,t+\Delta t} = u_{o,t} + \frac{4 r_e^2 \Delta T}{\Delta r^2}\left[\left(1+\frac{\Delta r}{2r}\right)u_{3,t} + \left(1-\frac{\Delta r}{2r}\right)u_{1,t} - 2u_{o,t}\right]$$

$\frac{r}{r_w}$ 1	2	3	4	5	T_h
0	100	100	100	100	0
0	62.5	100	100	100	0.005
0	62.5	84.4	100	100	0.010
0	52.7	84.4	93.2	100	0.015
0	52.7	76.3	93.2	93.2	0.020
0	43.8	64.3	78.9	78.9	0.040
0	26.4	38.8	47.7	47.7	0.100
0	11.5	16.9	20.7	20.7	0.200

$r_e = 5\ r_w$

$\Delta r = r_w$

$\Delta T_h = \frac{0.5(\Delta r)^2}{4\ r_e}$

Figure 12. Numerical Procedure for Consolidation by Radial Flow

requires the clay layer to be homogeneous if the analytical theories are to be used, or at least homogeneous in horizontal planes, and the variations in soil properties known in vertical direction, if the numerical procedures are to be used.

In addition, it is necessary that the values of the soil properties be established with a reasonable degree of accuracy. The coefficients of consolidation due to vertical and to horizontal flow of water, and the coefficients of permeability in the vertical and horizontal directions as well as for remolded samples, must all be determined from a large enough number of samples that a reliable average value of each is established. Since the need for sand drains and their effectiveness, if installed, are directly dependent upon these soil properties, it is evident that serious miscalculations could result if values of the soil properties are inaccurately determined and then used as the basis for design.

Use of Point Pore Pressures

In addition to settlement measurements which can be compared to theoretical predictions, the time rate of change of excess pore water pressure may be measured at specific locations within the clay layer. These pressure measurements can be compared with diagrams prepared by use of Eqs. (7) and (12) or (15) which predict the theoretical pressure-time behavior during consolidation. However, unless a good evaluation of the radius of the smeared zone at the well periphery and the coefficient of permeability within this smeared region were obtained, it is likely that a considerable difference may occur between the geometrical value of $n \left(= \dfrac{d_e}{d_w} \right)$ and the effective value of n which includes the effect of well smear. This will lead to appreciable variation in the predicted excess pore pressure vs. time relations.

Difficulties also arise in comparison of theoretical and field results which are partially due to the fact that the piezometer readings define the behavior of the clay at a point, or at least within a small volume, while the theoretical predictions are based, at best, on representative values of the soil properties. It would be expected that better agreement between actual consolidation behavior and that predicted from theory would be obtained by increasing the number of piezometer installations, thereby minimizing the effects of local variations in soil properties.

CONCLUSIONS

In the preceding pages the available theories for vertical consolidation due to vertical flow and due to radial flow of water to a drain well have been reviewed. These theories are entirely satisfactory for the analysis of any situation which conforms to the assumptions upon which these theories are based.

Including void ratio as a variable did not change the consolidation-time characteristics of vertical consolidations by vertical flow significantly. Thus, a consideration of variable void ratio does not contribute to the explanation of secondary consolidation.

For vertical consolidation due to radial flow toward a drain well, the "equal strain" solutions given by Barron are much more convenient to use than are the "free strain" solutions. Using Barron's equal strain solutions for ideal wells and for smeared wells, it was shown that the consolidation behavior of the latter is identical to that of an equivalent ideal well of reduced diameter. Diagrams are given for quantitative evaluation of this relation between behavior of smeared and equivalent ideal wells. By interpreting the consolidation behavior due to an actual well in terms of that of an equivalent ideal well, the numerous figures prepared by Barron for ideal wells can be used directly for design or analysis. An example shows the effectiveness of even a small diameter ideal well.

The numerical procedures for solving consolidation problems were found to be versatile aids for both checking the classical solutions and for evaluating new problems. Variable rates of loading, variable soil properties, layered systems, etc., can be readily included into the solution of consolidation problems by these methods.

ACKNOWLEDGMENT

This review of the theories of sand drains was prepared for Moran, Proctor, Mueser, and Rutledge, consulting engineers, as a part of their research project for the Bureau of Yards and Docks, Dept. of Navy, on the study and evaluation of sand drain installations. The author is grateful to the Bureau of Yards and Docks for their permission to publish the results of this portion of the study.

The author wishes to acknowledge particularly the valuable advice and suggestions given by Dr. P. C. Rutledge and Mr. S. J. Johnson, partner and associate, respectively, of the firm mentioned above. Other members of the firm have also contributed ideas and suggestions during the progress of this study.

Mr. R. L. Schiffman, now Assistant Professor of Civil Engineering, Lehigh University, checked the derivations of Barron's equations under the writer's supervision. This was carried out during the summer of 1955, when both Mr. Schiffman and the writer were temporarily employed by Moran, Proctor, Mueser, and Rutledge.

APPENDIX I

Definitions of Letters and Symbols Used

a_v = coefficient of compressibility

A = area, or $A = \dfrac{c_v \Delta t}{\Delta z^2} = \dfrac{k(1+e)\Delta t}{a_v \gamma_w (\Delta z)^2}$ in the numerical solutions

A' = $\dfrac{k \Delta t}{a_v \gamma_w (\Delta h)^2}$ = constant in numerical solution including variable void ratio

c_v = coefficient of consolidation $c_v = \dfrac{k_v(1+e)}{a_v \gamma_w}$

c_{v_r} = coefficient of consolidation due to radial water flow

$c_{v_r} = \frac{k_h(1+e)}{a_v \gamma_w}$

e = void ratio at any point in the soil layer

$\bar{e}$ = average value of the void ratio in the soil layer

$F(n)$ = $\frac{n^2}{n^2 - 1} \log_\epsilon (n) - \frac{3n^2 - 1}{4n^2}$ = function of n for equal strain solution

H = thickness of clay layer which is drained from one surface only, or half thickness of clay layer drained from top <u>and</u> bottom surfaces.

H_o = equivalent thickness of layer of solids in a clay layer of overall thickness, H

$J_o(\ )$ = Bessel function of the first kind, zero order

$J_1(\ )$ = Bessel function of the first kind, first order

k = coefficient of permeability (cm/sec)

k_h = coefficient of permeability in the horizontal direction

k_s = coefficient of permeability in the smeared zone

k_v = coefficient of permeability in the vertical direction

n = $\frac{r_e}{r_w} = \frac{d_e}{d_w}$ = ratio of diameter of well influence to diameter of drain well

p = $\bar{p} + u$ = total pressure at any point in the soil layer

$\bar{p}$ = effective, or intergranular, pressure at any point in the soil layer

r = radius

r_e = radius of influence of drain well

r_w = radius of drain well

r_s = radius defining boundary of smeared zone

s = $\frac{r_s}{r_w}$ = ratio of radius of smeared zone to radius of drain well

h = measure of distance ($dz = (1 + e)\ dh$)

t = time

Δt = time interval

T_v = $\frac{k_v (1+e)}{\gamma_w a_v} \frac{t}{\bar{H}^2}$ = dimensionless time factor for consolidation by vertical water flow

T_h = $\frac{k_h (1+e)\ t}{\gamma_w a_v d_e^2}$ = dimensionless time factor for consolidation by radial water flow

u = excess pore water pressure at a point in the clay layer

u_x = excess pore water pressure at a point in the clay layer as a result of water flow in the x-direction

u_r = excess pore water pressure at a point in the clay layer as a result of radial flow of water

u_z = excess pore water pressure at a point in the clay layer as a result of vertical flow of water

$u_{o,t}$ = excess pore water pressure at point o, at time t

$u_{2,t+\Delta t}$ = excess pore water pressure at point 2, at time $t + \Delta t$

$Y_o(\ )$ = Bessel function of the second kind, zero order

$Y_1(\ )$ = Bessel function of the second kind, first order

z = distance below the top surface of a clay layer

Δz = interval of z

$\alpha_1, \alpha_2, \alpha_3$ = roots of the Bessel function

γ_w = density of water (gm/cm^3)

ϵ = base of natural logarithms = 2.718----

ζ = $\frac{z}{H}$ = ratio of depth to a point to the thickness of the clay layer

λ = $\frac{-8T_h}{F(n)}$ = exponent for the "equal strain" solution

ν = $\frac{n^2}{n^2 - s^2} \log_\epsilon \left(\frac{n}{s}\right) - \frac{3}{4} + \frac{s^2}{4n^2} + \frac{k_h}{k_s} \left(\frac{n^2 - s^2}{n^2}\right) \log_\epsilon\ (s)$, factor in the solution for equal strain with smear

ξ = $\frac{-8\ T_h}{\nu}$ - exponent in solution for equal strain with smear

APPENDIX II

Derivation of Consolidation Equations Including Void Ratio as a Variable

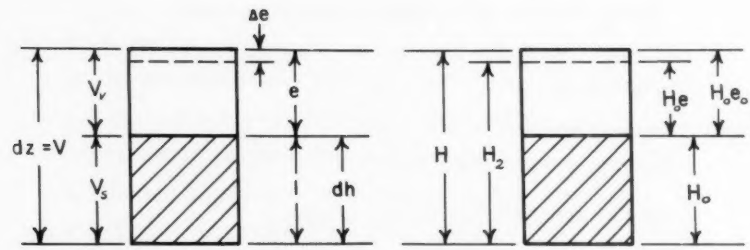
For a soil element of area A and height dz (Fig. 13) the volume of solids can be represented by a height dh which does not change during the consolidation process. The total height of the element may always be determined by the relation

$$dz = (1 + e)\ dh \tag{4}$$

The change of volume of a soil element is dependent upon the change of the volume of voids which is $V_v = Ae\ dh$. Thus the change of volume with time is

$$\frac{\partial V}{\partial t} = -A\ dh\ \frac{\partial e}{\partial t} \tag{35}$$

Next, considering the upward vertical flow of water through this elemental volume, an amount equal to $\frac{-k}{\gamma_w} \frac{\partial u}{\partial z}$ A flows through the bottom face and



(a.) For Unit Area and Height d_z

(b.) For Unit Area and Height H

Figure 13. Components of a Volume of Soil

an amount equal to $\frac{kA}{\gamma_w}\left(-\frac{\partial u}{\partial t} - \frac{\partial^2 u}{\partial_z{}^2}\ dz\right)$ flows through the top face.

The difference between flow at these two surfaces determines the rate of loss of water from the soil element as

$$\Delta Q = -\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2}\, A\, dz \tag{36}$$

Since it was assumed that the soil was completely saturated, and that both water and the solid soil particles are incompressible, the rate of water loss (Eq. (36)) must equal the rate of volume change of the soil element (Eq. (35)) or,

$$-A\, dh \frac{\partial e}{\partial t} = -\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2}\, A\, dz \tag{37}$$

In order to convert Eq. (37) into terms involving h, the following relations may be used.

$$\frac{\partial u}{\partial z} = \frac{\partial u}{\partial h}\frac{\partial h}{\partial z} = \frac{\partial u}{\partial h}\frac{1}{1+e} \tag{38}$$

$$\frac{\partial^2 u}{\partial z^2} = \frac{\partial h}{\partial z}\frac{\partial}{\partial h}\left\{\frac{\partial u}{\partial h}\frac{1}{(1+e)}\right\} = \frac{1}{(1+e)^2}\left\{\frac{\partial^2 u}{\partial h^2} - \frac{1}{1+e}\frac{\partial u}{\partial h}\frac{\partial e}{\partial h}\right\} \tag{39}$$

$$\frac{\partial \bar{p}}{\partial t} = -\frac{1}{a_v}\frac{\partial e}{\partial t} = -\frac{\partial u}{\partial t} \tag{40}$$

$$\frac{\partial \bar{p}}{\partial h} = -\frac{1}{a_v}\frac{\partial e}{\partial h} = -\frac{\partial u}{\partial h} \tag{41}$$

By substituting Eqs. (38), (39), (40), and (41) into Eq. (37) the expression is,

$$\frac{\partial u}{\partial t} = \frac{k}{a_v \gamma_w}\frac{1}{(1+e)}\left\{\frac{\partial^2 u}{\partial h^2} - \frac{a_v}{1+e}\left(\frac{\partial u}{\partial h}\right)^2\right\} \tag{42}$$

which is the general equation for consolidation with void ratio considered as a variable.

For solution of Eq. (42) by the explicit numerical scheme, it was found convenient to substitute terms for void ratio in place of those for u, according to Eqs. (40) and (41). The resulting difference equation is,

$$e_{o,t+\Delta t} = e_{o,t} + \frac{k\Delta t}{a_v \gamma_w (\Delta h)^2}\left[\frac{e_{2,t} - 2e_{o,t} + e_{4,t}}{1 + e_{o,t}} - \frac{(e_{2,t} - e_{4,t})^2}{4(1+e_{o,t})^2}\right] \tag{43}$$

Let

$$T = \frac{k\,t}{a_v \gamma_w (H_o)^2} \quad \text{or} \quad \Delta T = \frac{k\,\Delta t}{a_v \gamma_w (H_o)^2} \tag{44}$$

which is similar to T_v (Eq. 8) in appearance and is related by $T = T_v(1 + e_o)$ since $H = H_o(1 + e_o)$ as shown on Fig. 13.

Again,

$$\zeta = \frac{h}{H_o} = \frac{h(1+e_o)}{H_o(1+e_o)} = \frac{z}{H}, \quad \text{or} \ \Delta h = H_o \Delta \zeta \tag{45}$$

and

$$\frac{k\,\Delta t}{a_v \gamma_w (\Delta h)^2} = \frac{\Delta T}{(\Delta \zeta)^2} = A' \tag{46}$$

The curves shown on Fig. 1 were obtained by use of Eq. (43).

During consolidation, the void ratio changes with time from an initial value of e_o to a final value of e_2 which is reached at an infinite time. Thus e_o and e_2 are taken as the boundary values, and Eq. (43) determines the manner in which the void ratio changes with time.

The void ratio vs. time factor curve which results from using the boundary values of $e_o = 0.65$, and $e_2 = 0.55$ with Eq. (43) is shown as the solid line on Fig. 14, designated as "Including Effect of Variable Void Ratio." The other curve on Fig. 14 drawn as a solid line represents the results obtained from the Terzaghi theory if e_o and e_2 are considered to represent 0% and 100% consolidation, respectively. The dashed curve represents a typical consolidation test, converted into terms of e and T_v, which exhibits the region of "secondary" consolidation.

Secondary consolidation is characterized by a nearly linear relation between decrease in void ratio and logarithm of time, over a considerable period of time during the final portion of the test.

From Fig. 14 it is evident that the modification of the Terzaghi theory to include the consideration of variable void ratio does not appreciably change the void ratio-time factor relations during consolidation. The effect of variable void ratio is to throw the curve slightly to the left of the conventional curve. Thus the effect of including a consideration of variable void ratio in the theory of consolidation of a saturated soil does not contribute to the explanation of secondary consolidation.

By interpreting e_o and e_2 as the 100% and 0% values of excess pore water pressure, the results of the three solutions for variable void ratio can be plotted on the same diagram, as was done on Fig. 1.

REFERENCES

1. "Erdbaumechanik auf Bodenphsikalischer Grundlage" by K. Terzaghi. Vienna, F. Deuticke. 1925.

2. "Theorie der Setzung von Tonschichten" by K. Terzaghi and O. K. Fröhlich. Vienna, F. Deuticke. 1936.

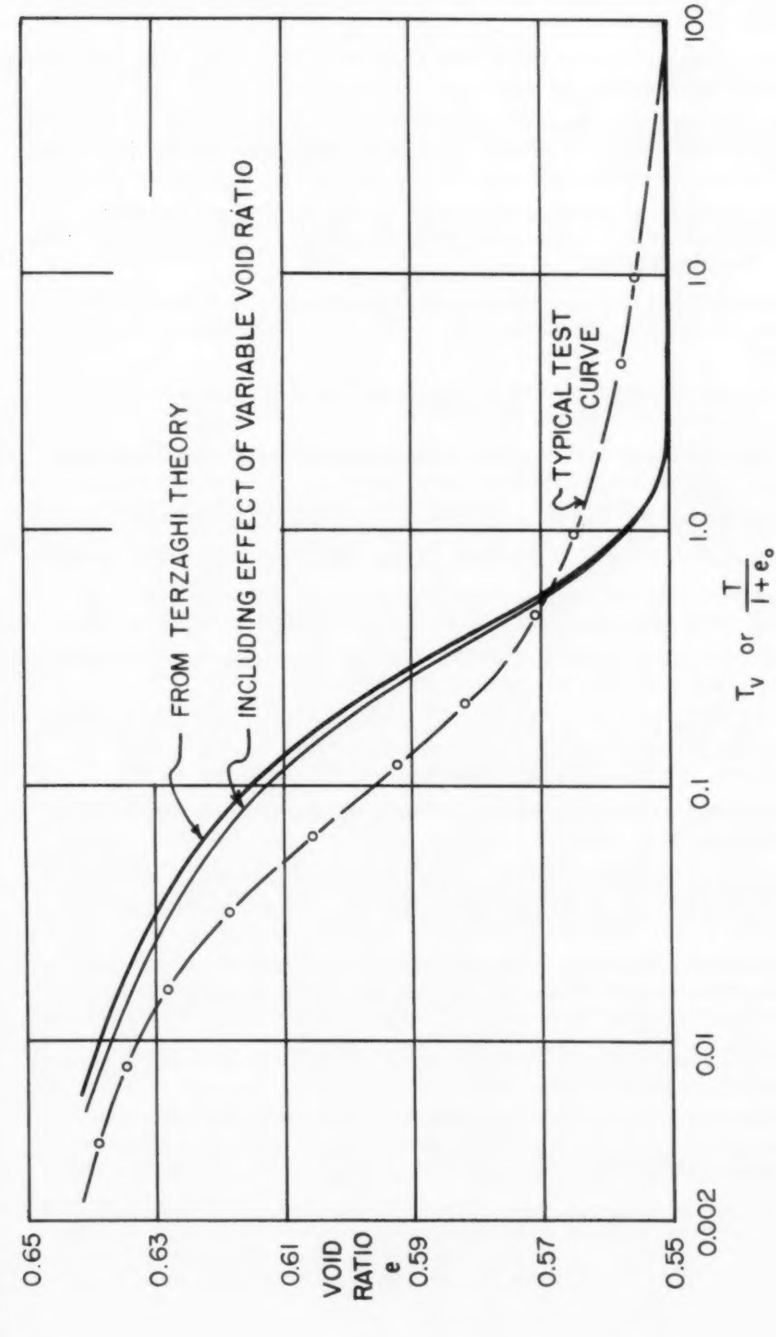


Figure 14. Void Ratio — Time Factor Relations

3. "Theoretical Soil Mechanics" by K. Terzaghi. John Wiley & Sons. New York, N. Y. 1943.

4. "The Drying of Porous Solids" by A. B. Newman. Trans. Am. Inst. Chem. Eng. Vol. 27 p. 310, 1931.

5. "Simple Two- and Three-Dimensional Cases in the Theory of Consolidation of Soils," by N. Carrillo. J. of Math. and Phys. Vol. 21, No. 1. Mar. 1942.

6. "Der hydrodynamische Spannungsaugleich in zentral entwässerten Tonzylindern," by L. Reudulic, Wasserwirtsch. u. Technik. Vol. 2. 1935. pp. 250-253, 269-273.

7. "The Influence of Drain Wells on the Consolidation of Fine Grained Soils" by R. A. Barron. Providence (R. I.) District. U. S. Engineers Office. 1944.

8. "Drainage of Clay Strata by Filter Wells." by K. Terzaghi. Civil Engineering October 1945 pp. 463-464.

9. "Consolidation of Fine-Grained Soils by Drain Wells" by R. A. Barron Trans. ASCE vol. 113, 1948.

10. Tech. Memo. No. 158 by R. E. Glover. Bu. Recl. Denver, Col. 1930.

11. Disc. of Ref. 10 by W. Kjellman. Trans. ASCE Vol. 113. 1948. p. 748.

12. "Soil Mechanics in Engineering Practice" by K. Terzaghi and R. B. Peck. John Wiley and Sons. New York, N. Y. 1948 (Fig. 29 p. 77)

13. "The Numerical Solution of Heat Conduction Problems" by H. W. Emmons Trans. A.S.M.E. Vol. 65, 1943 pp. 607-615.

14. "A Practical Method for Numerical Evaluation of Solutions of Partial Differential Equations of the Heat-Conduction Type" by J. Crank and P. Nicolson. Proc. Cambridge Phil. Soc. Vol. 43, 1947. pp. 50-67.

15. "Numerical Analysis of Heat Flow" by G. M. Dusinberre. McGraw-Hill. New York, N. Y. 1949.

16. "A Study of the Numerical Solution of Partial Differential Equations" by G. G. O'Brien, M. A. Hyman, and S. Kaplan. Journal of Math. and Physics V. 29. 1950 pp. 223-251.

17. "Numerical Solution of Some Problems in the Consolidation of Clay" by R. E. Gibson and P. Lumb. Proc. Inst. of Civil Engineers v. 2 Part I. 1953. pp. 182-198.

18. "Numerical Analysis of Consolidation Problems" by R. F. Scott. Unpublished M. S. Thesis. MIT. 1953.

19. "A Method of Numerical Integration for Transient Problems of Heat Conduction" by T. P. Tung and N. M. Newmark. Univ. of Illinois Civil Engineering Studies. Structural Research Series No. 95. March 1955.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

DETERMINATION OF THE 0.02 mm FRACTION IN GRANULAR SOILS

R. W. Johnson,[1] A.M. ASCE
(Proc. Paper 1309)

ABSTRACT

This paper describes the method of using statistical analysis to determine the constant representing the ratio of the percent of material finer than .02 mm. to the percent passing the No. 200 sieve. Results and accuracies of tests on twenty-four granular soils are given, together with values for the constant.

INTRODUCTION

Although there are many physical factors of soil which influence its susceptibility to frost action, particle size and gradation are among the more important since they directly affect the moisture content capacity and capillarity of the soil.

In 1931 Casagrande advanced the theory based on test results that the particle size of .02 mm. was significant in regard to frost action in soils. On the basis of his studies he concluded that for non-uniform soils containing more than 3% of grains smaller than .02 mm. considerable ice segregation could be expected to occur. For uniform soils, more than 10% of grains smaller than .02 mm. was considered critical.

The Corps of Engineers used this information in further investigations of frost susceptibility. Although some soils with as much as 18% finer than .02 mm. did not show detrimental frost action, the report concluded that "the presently used criterion which states that well graded soils with less than 3%, by weight, finer than .02 mm. are not frost susceptible, has proven a useful rule but other factors, such as the character of the fines, must be considered in recognizing frost susceptible soils with accuracy or in predicting the intensity of ice segregation which may be expected."

Some agencies, in using this criteria, have failed to take into

Note: Discussion open until December 1, 1957. Paper 1309 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83, No. SM 3, July, 1957.

1. Eng. Div., USAREUR Const. Agency, Germany.

consideration the various characteristics of the soils, their gradation and physical properties, and have used the 3 percent rule as a standard specification in order to preclude the use of frost susceptible material where it may be damaging.

In particular this rule has been applied to certain toll road construction where it was included in the requirements for base material for concrete paved road construction.

The test specifications covering the construction on this project were standard AASHO specifications which employ the use of the No. 200 sieve to separate silt and sand, all material passing the No. 200 being classed as silt and clay.

Since the particle size of .02 mm. is an in-between silt size, it cannot be separated by any normal test methods except the hydrometer analysis.

In actual practice the determination of the amount of material finer than .02 mm. for the gravelly sand base material was quite laborious, involving the complete hydrometer test. This writer simplified the test somewhat by setting up certain standard test conditions, solving Stokes Law for the time required for all material larger than .02 mm. to leave the suspension and ran the test using this one point determination. This proved to be a considerable improvement over the standard hydrometer analysis.

After many such tests had been run it was reasoned that it appeared that the slope of the granulometric grain size curve for the finer fractions and for similar soils was very similar. This seemed to indicate the possibility of relating the .02 mm. fraction to the fraction passing the No. 200 (.076 mm.) sieve. If this could be done, it should be possible to find a constant which would express this relationship providing the slope was the same for similar soils.

In this paper are described the method used to determine this relationship, the statistical analysis of the results, and the final conclusions regarding the study.

Laboratory Operations

With the cooperation of the Minnesota Highway Department, 24 samples of granular soils used in road construction during the year 1954 were obtained.

These soils were portions of the original test samples used by the Highway Department for laboratory tests. These soils had been sieved and the amounts passing the No. 4, 10, and 200 sieves were known (Table 1). It was attempted to obtain a variety of amounts passing the No. 200 sieve. This amount varied from 5.3% to 12.7%.

The individual samples were prepared for the standard (T88-42) AASHO hydrometer analysis test, and the test run. One hundred gram samples were used to give a larger percent of fines in suspension. The results were calculated and the granulometric curve drawn for each soil.

In Table 2 a comparison between the laboratory analysis and the Minnesota Highway Department tests is shown for the minus 200 fraction. It will be noted that some differences occur between the two results. Since the laboratory result is representative of the actual sample condition, it will be used for the calculations which follow. It is believed that the combination of handling and worn cloth bags which held the sample caused mutilation of the sample and hence the different results.

The accuracy of the final results is dependent on the accuracy of the hydrometer analysis itself. In order to determine what accuracy could be expected the test was repeated and the sample was again dispersed by shaking for one minute; the 1, 2, 5, and 15 minute readings being recorded. These too, were computed and plotted as pencil dots on the grain size curve. The amount of .02 mm. material was determined by lining up these points with a french curve and reading the percentages. The points were not connected by a permanent line in order to avoid confusion since the curves plot very close to each other. In drawing the grain size curve it was attempted to connect the points as accurately as possible in the .02 mm. area. This aided in reading the percentage of .02 mm. material and assisted in obtaining accuracy. The percent of material passing the No. 200 sieve was obtained by calculation and is the value used in subsequent calculations. It is not the result obtained from the grain size curve since the curve does not intersect the plotted point in all cases.

Analysis of Results

After the amount of material passing the No. 200 sieve and the amount finer than .02 mm. was known, the percent of minus .02 mm. material was related to the percent passing the No. 200 sieve and recorded in Table 2 as a decimal.

In order to analyze these numerical results by statistical method it must first be determined if the data are adequate and also if they are correct.

Since these results vary according to magnitude, they were first arranged according to size or magnitude to form a frequency distribution (Table 3). Using the range of the data as a guide, the data were arranged into a series of six groups which are known as a class interval. These groups are as follows.

Interval	No. in each interval
.1 - .19	1
.2 - .29	7
.3 - .39	9
.4 - .49	4
.5 - .59	2
.6 - .69	1

When plotted to form a frequency distribution curve as shown in Fig. 1, the data plot to form a true curve. Its lack of symmetry or skewness to the right indicates that it is distorted by extremes in the higher values.

This initial manipulation of the data indicates that it can be analyzed statistically even though appearing on the surface to be rather meager.

An average can be described as a typical value which tends to sum up the mass of data. It serves as a basis for measuring or evaluating extreme or unusual values and to measure the location of central tendency.

Several kinds of averages are in common use, the most important of which are:

1. The arithmetic mean
2. The median
3. The mode.

The ease of computation and long usage make the arithmetic mean the best known and most commonly used. It can be expressed by formula as

$$M = \frac{\Sigma(X)}{N}$$

where: M = arithmetic mean
Σ = "sum of"
X = data expressed as individual items
N = number of items

For the data used in this report which is found in Table 3, the arithmetic mean is

$$M = \frac{\Sigma(X)}{N} = \frac{8.918}{24}$$

$$= .372$$

Since the value of the arithmetic mean is determined by every item in the distribution, its value is distorted by extreme values and may not be typical.

A better value for the average of a mass of data is the median. It is defined as the value of the middle item when the items are arranged in order of magnitude. For the case under study which has an even number of items, the average of the middle two is the median. The median of this data under study is shown to be the average of items 12 and 13 (Table 3) and is found to be .353.

Since the number of items controls the value of the median, it is not affected by extreme values in either the high or low ranges. It is not as familiar as the arithmetic mean and is not so widely used, however. It can be seen that it is a more typical value of the series because of its independence of unusual values.

The mode, by definition, is the most frequent or common value of the series; provided a sufficiently large number of items are available to give a normal distribution (See Fig. 1).

It is not possible to calculate mathematically the exact value of the mode. It has been found empirically, however, that the distance between the mean and the median is one-third of the distance between the mean and the mode.

OR: mode = mean - 3 (mean - median)

For this series:

$$\begin{aligned} \text{mode} &= .372 - 3\,(.372 - .353) \\ &= .372 - .057 \\ &= .315 \end{aligned}$$

The mode is also an average of position and is independent of extreme values. Since it is the most typical it is also the most descriptive average.

In order to check the accuracy of the hydrometer test results, the repeat tests, where made, were used to compute the arithmetic mean and the median. This gave the values of

1. arithmetic mean = .371
2. median = .360

This shows that the arithmetic mean differs very little from the original

value (.371 compared to .372) and the median changes from an original value of .353 to .360. This close comparison is considered to be good.

The average or typical value is of little use unless the degree of variation which occurs about it is known. For the data under study the deviation will be relatively large since the range (difference between the maximum and minimum values in the series) is large and the number of items is not great. However as the frequency distribution curve shows, the data follow a normal distribution and the values for the means would not vary appreciably, even were the number of items (tests) increased considerably.

The mean deviation can be computed about either the arithmetic mean or the median with the deviation from median being a minimum. It is given by formula as

$$MD = \frac{\Sigma(d)}{N}$$

where:

MD = mean deviation
Σ = "sum of"
d = the deviation of each value from the mean or median (signs ignored)
N = number of items

Using the arithmetic mean, .372, (Table 3) the mean deviation is found to be:

$$MD = \frac{\Sigma(d)}{N} = \frac{2.131}{24} = .0888$$

Using the median, .353, (Table 3) the mean deviation is:

$$MD = \frac{\Sigma(d)}{N} = \frac{2.092}{24} = .0872$$

The percent of variation from the arithmetic mean is

$$\frac{.0882(100)}{.372} = 24\%$$

The percent of variation from the median is

$$\frac{.0872(100)}{.353} = 25\%$$

These values are quite large but as explained before, the number of items controls the value of the deviation from the central tendency. One can conclude from this test that these values of percent of finer than .02 mm. material computed from the amount passing the No. 200 sieve are correct within 25% plus or minus.

CONCLUSIONS

From the results of this study the following conclusions are made:

1. As indicated by the grain size analysis curves and the repeat tests, the hydrometer analysis gives accurate values for the textural analysis of soils including granular types.
2. By statistical analysis, the amount of material finer than .02 mm. can be computed from the amount of material passing the No. 200 sieve by multiplying the latter by a value of .353 or .372 with .360 (the median from repeat tests) being an acceptable constant. These constants give values accurate within 25%.
3. For similar granular soils, this method is considered to be acceptable within the accuracy indicated, however, further tests using more and varied soils are suggested.

REFERENCES

1. Casagrande, A. "Discussion on Frost Heaving." Proceedings, Highway Research Board, Vol. 11, Pt. 1, pp. 168-172, 1931.
2. Haley, James F., and Kapler, Chester W. "Cold Room Studies of Frost Action in Soils," Symposium on Frost Heave and Frost Action in Soil, Highway Research Board, Special Publication, pp. 246-267, 1951.
3. "Statistical Methods," by H. Arkin and R. R. Colton, Barnes and Noble, New York, 1939.

TABLE 1

MINNESOTA HIGHWAY DEPARTMENT
SIEVE ANALYSIS RESULTS

Test No.	M.D.H. No.*	Percent Passing						
		#4	#10	#20	#40	#80	#100	#200
1	2725	74.0	60.4	39.6	23.6	9.8	8.5	6.8
2	2759	65.9	61.0	51.6	37.7	18.5	15.0	9.0
3	2791	81.0	72.7	57.4	36.2	12.1	9.8	6.8
4	2709	62.4	55.9	39.9	15.6	6.6	6.0	4.7
5	2786	75.3	61.2	47.0	35.4	19.3	17.2	11.6
6	2774	85.2	77.5	66.6	48.4	14.1	10.5	5.2
7	2761	65.5	58.4	48.8	36.0	21.3	17.9	10.5
8	2724	71.8	58.8	40.7	24.2	10.4	9.1	7.2
9	2826	80.1	72.3	59.9	46.0	22.9	18.4	11.0
10	2748	63.3	54.6	43.5	30.9	18.2	15.6	9.7
11	2749	63.8	55.4	43.2	30.6	17.1	14.6	9.2
12	2889	56.9	46.5	33.3	21.5	18.2	14.0	10.7
13	2892	71.3	62.3	46.9	25.0	10.3	9.2	7.4
14	2857	83.9	72.2	55.3	35.9	13.3	9.8	5.6
15	2755	63.5	52.0	37.9	26.3	15.4	13.1	9.0
16	2832	64.2	54.9	40.8	26.6	13.1	11.0	6.8
17	2772	62.4	51.4	38.0	24.7	12.3	10.4	7.0
18	2746	69.8	60.5	44.8	31.1	19.2	16.5	10.0
19	2827	94.9	92.6	85.7	70.2	34.9	28.2	12.7
20	2856	52.6	44.0	33.2	20.7	8.9	7.6	5.7
21	2711	61.2	52.9	40.6	22.6	10.3	9.4	7.6
22	2877	63.7	54.0	39.8	26.3	13.7	11.8	7.8
23	2833	67.0	57.2	44.4	30.0	14.2	11.5	7.1
24	2806	72.9	65.5	51.0	36.4	14.3	9.9	5.3

* Minnesota Highway Department

TABLE 2

SUMMARY OF RESULTS OF THE HYDROMETER ANALYSIS TESTS

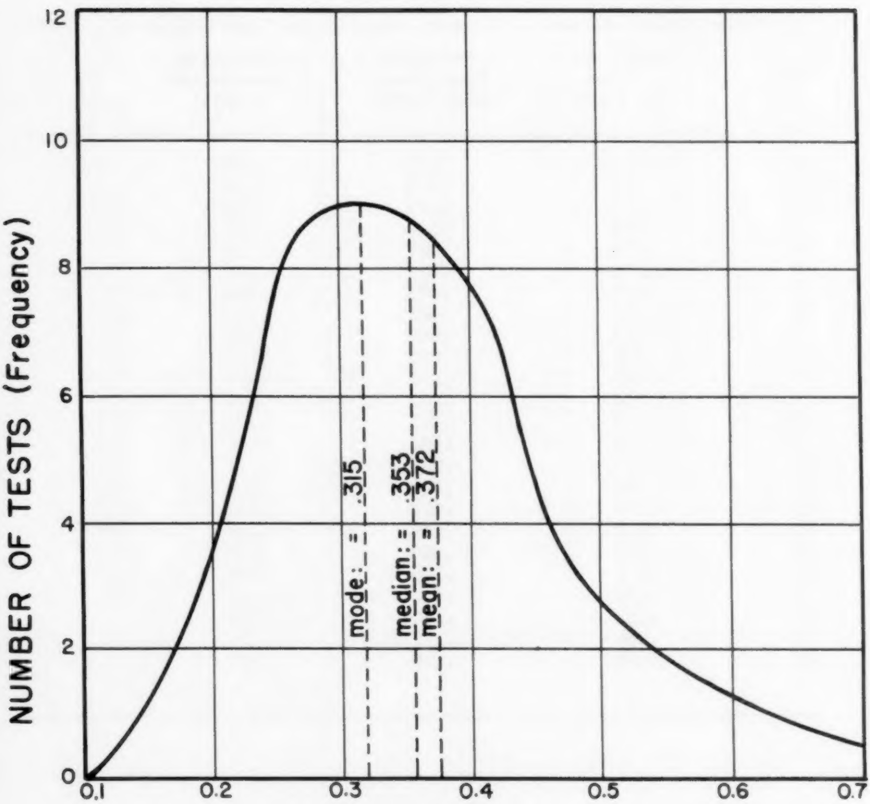
Sample No.	Percent Pass # 200	M.H.D. Percent Pass # 200	Percent .02 mm.	Repeat Percent .02mm.	Ratio of .02mm. to Percent Pass #200	Repeat Ratio of .02 mm. to Percent Pass #200
2774	7.1	5.2	4.0	2.9	.564	.408
2791	8.0	6.8	3.7	3.5	.462	.438
2833	6.4	7.1	1.5	1.9	.234	.297
2827	17.1	12.7	3.0	3.0	.175	.175
2826	12.2	11.0	4.6		.377	
2889	8.1	10.7	2.7		.334	
2724	7.9	7.2	4.2		.532	
2761	10.9	10.5	3.0		.275	
2749	10.5	9.2	2.8		.267	
2748	10.3	9.7	3.0		.291	
2786	11.1	11.6	3.9		.351	
2709	6.2	4.7	3.0		.484	
2772	8.0	7.0	3.0	3.0	.375	.375
2711	8.4	7.6	3.2	3.2	.381	.381
2877	7.6	7.8	2.7	2.4	.355	.316
2759	9.7	9.0	2.8	2.6	.289	.268
2755	7.2	9.0	3.3	3.3	.458	.458
2725	7.6	6.8	5.0	4.3	.658	.567
2832	6.5	6.8	2.0	2.1	.308	.323
2746	11.1	10.0	3.1	3.3	.279	.298
2806	7.3	5.3	2.5	2.7	.342	.370
2856	5.6	5.7	1.5	1.7	.268	.304
2857	5.5	5.6	2.1	2.6	.382	.473
2892	8.8	7.4	4.2	4.7	.477	.534

TABLE 3

COMPUTATION OF MEAN DEVIATION

Item No.	Ratio of % .02mm. to % #200	Deviation From Arith. Mean (.372)	Deviation From Median (.353)
1	.175	.197	.178
2	.234	.138	.119
3	.267	.105	.086
4	.268	.104	.085
5	.275	.097	.078
6	.279	.093	.074
7	.289	.083	.064
8	.291	.081	.062
9	.308	.064	.045
10	.334	.038	.019
11	.342	.030	.011
12	.351	.021	.002
13	.355	.017	.002
14	.375	.003	.022
15	.377	.005	.024
16	.381	.009	.028
17	.382	.010	.029
18	.458	.086	.105
19	.462	.090	.109
20	.477	.105	.124
21	.484	.112	.131
22	.532	.160	.179
23	.564	.192	.211
24	.658	.286	.305

FREQUENCY DISTRIBUTION CURVE



CLASS INTERVAL
(Percent .02mm material of - # 200 fraction)

Fig. 1

Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1319)

(Over)

Note: Paper 1319 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83 SM 3, July, 1957.

Discussion of
"EXPERIENCES WITH LOESS AS FOUNDATION MATERIAL"

by W. A. Clevenger
(Proc. Paper 1025)

W. A. CLEVENGER,[1] A.M., ASCE.—The discussions of Messrs. Cedergren, Peck and Ireland are appreciated and contribute substantially to furthering the knowledge of the engineering behavior of loess. Mr. Cedergren has pointed out an important characteristic of loess which must be considered, particularly in the design of hydraulic structures, namely, that of low erosion resistance. Many miles of lined and unlined canals and drainage channels have been constructed in the Missouri Basin loess area. Because of its susceptibility to erosion, the slopes exposed to the weather are, of necessity, constructed as steep as possible to minimize such erosion (1/4:1 in many cases). The longitudinal slopes of unlined canals and channels are also controlled largely by the susceptibility of loess to erosion, whether in the undisturbed or disturbed condition.

In their discussion Messrs. Peck and Ireland expressed a question with regard to the sequence of rainfall, loading, and the settlement of the Kansas elevator. The words "July" and "August" were used in general terms in the original paper. It is agreed that some of the settlement occurred in July, and with the bin loading. However, it may be seen that considerable rainfall took place in July. The drainage conditions, which led to ponding between the north side of the building and the adjacent spur railroad track, existed throughout the period of settlement. Therefore, any heavy rainfall in the drainage area would cause an accumulation of water at the site. The fact that the tilting was later corrected to a considerable extent by artificially wetting the opposite side of the elevator foundation, further supports the thinking that the soil structure collapsed from wetting as well as from loading.

Messrs. Peck and Ireland have expressed the opinion that there is no correlation between density and supporting capacity of loess. Data are presented in their discussion to support this. These data show the relation between (1) density and load test results, and (2) density and consolidation test results for loess tested at the natural water content. For this moisture condition, the writer agrees with the data presented, as indicated by Figure 2 of the writer's paper. Experience, however, with structures which have suffered foundation failures on loess shows that they are almost always associated with foundation wetting. This leads to the belief that the ultimate supporting capacity is related to the ultimate settlement to be expected upon wetting. This, of course, is particularly true for loess foundations of hydraulic structures which are certain to become wetted. The discussers will probably agree that there is excellent correlation between supporting capacity and density if it is assumed that the loess will become thoroughly wetted.

The discussers' comment on the disturbance of loess csamples caused during normal "undisturbed" sampling operations is certainly well taken and

1. Chf. Engr., Woodward, Clyde and Associates, Oakland, Calif.

any possible misconception in this respect should be corrected. However, the writer intended the word "undisturbed" to apply to truly undisturbed samples taken by any method which ingenious men might devise.

Discussion of
"REDRIVING CHARACTERISTICS OF PILES"

by Nai-Chen Yang
(Proc. Paper 1026)

NAI-CHEN YANG,[1] M. ASCE.—The contributions of Messrs. Eremin, Zegarra and McNeill in discussing the writer's paper are greatly appreciated. As Mr. Eremin agreed on the general concept of redriving test, he suggested to extend its implication for detecting the thixotropic properties of soils. It is a fact that thixotropy has some contribution in effecting the redriving resistance of a pile. However, the change of pore-water conditions has been considered as the primary cause. The writer declines to speculate the thixotropic properties on the ground of the redriving resistance of piles.

The writer appreciates Mr. Zegarra's comments and his analysis on the practical application of redriving concept. He pointed out that for one job case, the computed resistance by equation (2) was 40% higher than that obtained by the load test. Consequently, he warned to those who indiscriminately would use it (the redriving concept). In reviewing Mr. Zegarra's presentation, the writer found that there were some mistakes in Mr. Zegarra's computations. First of all, the value of Q_o is equivalent to the total weight for setting the pile. The weight of soil and water, displaced by the pile, therefore, constitute the negative component of Q_o-value. In Figure 5, an itemized computation has been illustrated. In Mr. Zegarra's case, the appropriate value of Q_o is about 11,300 lbs. For a redriving resistance of 16 to 18 blows per ft., the ultimate resistance, Q, according to the equation (2), ranges from 26 to 28 1/2 tons or 4 to 14% higher than that obtained by the load test. Because of the extra-polation on observing results, the writer will consider such variation to be tolerable in foundation practice. Although Mr. Zegarra and his client have satisfied the pile capacity by its redriving test, the writer would advise them that the computed resistance by equation (2) represents only the ultimate capacity of a single pile, which is different from the resistance in a pile group.

The writer extends his gratitude to Mr. McNeill's comments on the performance of timber pile. His advanced analysis will be very helpful on this interesting subject. With reference to test pile No. 1, (See Fig. 8), the transitional load, from elastic to plastic state of equilibrium, is likely to be 90 tons. For the ultimate resistance at pile driving, the plastic state of equilibrium should be considered. The writer remains of the opinion that the failure load of test pile No. 1 is about 120 tons.

1. Foundation Engr., Madigan - Hyland, New York, N. Y.

Discussion of
"FOUNDATION STUDIES FOR DE LONG PIERS"

W. G. Shockley and T. B. Goode
(Proc. Paper 1080)

W. G. SHOCKLEY,[1] M. ASCE and T. B. GOODE.[2]—The authors appreciate the contribution of Mr. Focht in his more detailed analysis of the data presented in the paper. Even though the general relationship between computed and observed penetration resistances of the caissons appear reasnonable, there is sufficient lack of agreement (for example, the discusser's fig. 4) for the authors to recommend conservatism in the computation of caisson bearing capacities. A conservative approach to the computation of bearing capacity is also indicated when probable variations in soil strength and stratification between boring locations and the actual caisson locations are considered; this is, of course, a problem encountered frequently in soil mechanics. It is to be hoped that further studies along similar lines will assist in determining proper methods of evaluating bearing capacities of caissons in clays.

Mr. Focht mentions that the maximum load carried by the caissons may be greater than the load utilized to install it. This may be true of certain types of drilling platforms. However, it should be noted that in the case of the large DeLong piers additional weight can be applied to the caissons (up to the capacity of the air jacks) by jacking only a few caissons at a time or by pumping water into the barge; in this manner the installation loads on the caissons can be made greater than operational loadings for most military applications.

The authors agree that undisturbed sample borings and tests on samples obtained therefrom are probably the best means of securing data for determining caisson bearing capacities. Nevertheless, in a military application, speed of installation, the need for simple and rugged exploration equipment, and the lack of highly skilled personnel dictate the use of expedient exploration and evaluation techniques. Furthermore, the basic requirement for estimating caisson bearing capacity, and corresponding penetration, is to insure that available caisson lengths are adequate to insure raising the barge to its required height. It was on these bases that the authors favor the dynamic cone for field use.

1. Chief, Embankment and Foundation Branch, Soils Div., U. S. Army Engr. Waterways Experiment Station, Vicksburg, Miss.
2. Chief, Inspection and Exploration Section, Embankment and Foundation Branch, Soils Div., U. S. Army Engr. Waterways Experiment Station, Vicksburg, Miss.

Discussion of
"SEEPAGE THROUGH FOUNDATIONS CONTAINING DISCONTINUITIES"

by Elbert E. Esmiol
(Proc. Paper 1143)

JUDSON P. ELSTON,[1] M. ASCE.—This paper makes a valuable contribution to our studies and working knowledge of soil mechanics and foundations. While it is realized that depositional, erosional, and structural discontinuities are often masked by superficial geological conditions particularly before construction starts, the writer does not understand the author's premise that such deficiencies cannot be ascertained prior to completion of construction. Admittedly, where such conditions may be anticipated, an intensive, and sometimes expensive program of exploration concurrent with construction operations is necessary. In some cases, design may have to be altered or a program of remedy developed. As Mr. Esmiol states, "The degree of reliability of accepted methods for designing for seepage depends almost entirely on the accuracy with which foundation conditions have been delineated." It is the writer's opinion that, with an extensive and thorough pre-construction exploration consisting of the available or known methods of field determination and with sufficient laboratory work, most of these variations can be determined. In the rare or exceptional case where such defects went undiscovered during investigation, and the writer would place the author's case history of Enders Dam in this category, was it not possible to have determined the sand lenses, the thalweg of the ancient channel, by continued investigation during the construction stage? One should not misunderstand this question! There is no thought of criticism of individuals or organizations intended whatsoever. The writer's thought is whether a more complete program of investigation and research could not be carried out in the pre-construction stage. Again, it is true that more time and money would have to be expended before final design was embarked upon. Would it not be a more economical approach in the long run? In his paragraph on "Foundation Discontinuities," the author states that with careful study of site conditions, anomalies can be satisfactorily classified for reference and identification purposes.

Mr. Esmiol says that "the possibility of seepage assumes the status of a calculated risk." The writer agrees especially in the case of Granby Dikes Nos. 1, 2 and 4 with which he was personally acquainted. The writer is still of the opinion that the exploration program, at the time the cutoff trenches were opened and in the case of No. 1 widened and deepened, could well have included a series of diamond-drilled and cored grout test holes throughout the length of the trenches. The water passageways would have been completely delineated and an effective water barrier established. Certainly the cost of such a program during the construction stage would have been nominal regardless of the interpretation of the results, and the cost of the grouting work infinitesimal compared with the costs of such work after the

1. Asst. to Constr. Mgr., Power Area, Uhl, Hall & Rich, Engrs., St. Lawrence Power Project, Massena, N. Y.

reservoir was raised. The writer is inclined not to agree that " a thorough exploration program will not delineate all the characteristics of a foundation, and that surface manifestations when applied to subsurface conditions may be misleading." Nevertheless, Mr. Esmiol is to be highly complimented on his meticulous and thorough treatment of the subject material at hand. He has left no stone uncovered in his description of the various foundations and has demonstrated a remarkable ability and understanding of the problems involved in soil mechanics.

HOWARD F. HOFFMAN,[1] A.M. ASCE.—Foundation investigation for dams alone is insufficient. The area to be flooded should be subjected to extensive and intensive exploration to determine its physical, chemical and biochemical characteristics and possible modifications of the same by the presence, and pressure, of water. Truly the dam must be static, but should leakage occur elsewhere, though not calamitous, the value of the structure may be depreciated.

Discontinuities and faults should frequently be considered as both results and causes of earth movements. What modifications may occur during these diastrophisms are partially shown by past records. What will occur in future tremors should be considered in the light of all existing knowledge.

Wickiup Dam

Recorded experience in Oregon indicates that caution should be exercised in the vicinity of the Columbia River Basin. Each of us may take a good lesson from the leakage occuring at this reservoir. Elbert Blackwelder in Regional Geology of the United States of North America states, "-- for many reasons the geologic history of the Columbia River Basin is still imperfectly known." The construction of a reservoir in such a geologic environment should, therefore, be considered only after we have complete information. Without that we can reach no logical conclusion. The pumice covering of this area served to impede access to such complete information.

A basalt base should cause considerable concern. The high permeability of open joints and scoriacious contact zones together with the possibility of hidden canyons or coulees of ancient streams buried by evident volcanic, perhaps not tectonic, activities of the past might offer fair warning that we cannot reach a satisfactory conclusion without a quite complete knowledge of subsurface conditions.

The vesicular nature of basalt, combined with jointing, might offer fair warning that leakage is possible. Sediments covering such a base indicates need for a complete water table survey of the area. The character of cementing materials used by nature and the extent of the faults present might help to determine if seepage will be evident or is possible.

Preoperational investigations may not always furnish sufficient data as we proceed and practice since all contributing factors may not be properly interpreted, or connected, by any one of us.

1. Senior Public Health Engr., Utica, N. Y.

Granby Dikes 1, 2, and 4

The typical log given on 1143-15 indicates siltstone, overlain by clay, siltstone, sandstone, broken sandstone and jointed sandstone. The text indicates obliteration of numerous faults by glacial activity that developed at least three channels, later filled. During previous periods three lakes have been present.

Jointed sandstone, broken sandstone, and sandstone are the result of lack of water flooding an area or water over them. Which was it in this, or these, cases? In my own opinion sandstone may provide a nice building finish, but generally is good only to prevent the scouring of mechanical devices by loose sand. The sandstones may be the result of previous occupancy of the area by water. Their condition here shows considerable porosity. We do not know the exact cementing material and therefore cannot surmise that they are insoluble. Reason might lead one to believe them soluble first. That two-phase, double, condition Sand-water, Sand-air probably led to their formation which, wetted again, tended to revert to porous sand.

On the basis of the information presented it is assumed that Dike Number One was placed on jointed sandstone about three feet thick, overlying about five feet of very broken sandstone overlying sandstone, etc. This may have been done because "The North Park was believed to be sufficiently competent to support the dam, and impervious enough for an adequate water barrier." Were the constituents of this soil, sandstone, investigated? For how long can a layer of sandstone, over a layer of broken sandstone be considered impervious? Can the presence of previous stream channels, overlain by possibly previous materials, or materials which can be made pervious, in time, by water under pressure be considered tight?

Enders Dam

Nebraska presents a geologic picture which we must carefully consider. According to E. H. Barbour, the Eocene series and Triassic and Jurassic systems are missing. Beds of diatomaceous earth of some extent, and of a thickness varying from a few inches to five or six feet are known in the central counties of Nebraska, particularly on the region of the Loup System."(1) Might this not be a slight remnant of the Reptile, more specifically Ammonite, age.

Volcanic dust has also been sampled in Keya, Paya and Cherry counties to mention a few in the area of this dam. Such peculiar geologic conditions, added to porous material filling buried stream channels leads on to agree with Mr. Elbert E. Esmiol that a cutoff trench must completely penetrate all pervious materials to be a cutoff. The question of whether the "cutoff trench" should extend around the reservoir might also be considered.

McMullan Reservoir

Engineering News, 54-1-9, (7/6/'05) states that the original Earth Fill Dam had a maximum height of fifty-two feet with a crest length of one thousand six hundred and eighty-six feet (1,686). The crest has now been extended to twelve thousand and nineteen feet (12,019), in the neighborhood of seven hundred and thirteen percent (713%). The crest height has been lowered four feet,

to forty-eight feet. Whether by reason or by deposition has not been determined.

It is not surprising that leakage is increasing at this reservoir after looking at the Geologic Map of New Mexico, modified polyconic projection, prepared by N. H. Darton in 1928. This map shows alluvium deposits, Rustler Limestone, Casite Gypsum, and fine textured white, possibly permeable, sandstone underlying the area.

In addition a fault is located in the flooded area. Thus, we know the mountains moved, within some time before the flooding. Because of such bedding, coupled with the crest length extension of the dam, it is fair to conclude that "Such a condition resulted in progressively higher water loss." Southerly of this reservoir Carlsbad Caverns provides some indication of groundwater conditions in the area.

ACKNOWLEDGMENTS

My thanks are extended to the Geology Departments of Utica College of Syracuse University, Hamilton College and Colgate University for their help and forebearance in my excursions thru their domains.

REFERENCES

1. Nebraska Geological Survey, Vol. 1, E. H. Barbour, 1903.
2. Elements of Engineering Geology, H. Reis & T. L. Watson, John Wiley & Sons 1951-2nd Ed.
3. Application of Geology to Engineering Practices, R. M. Field, The Geological Society of America, Part I, E. B. Burwood, Jr.
4. Principles of Engineering Geology and Geotechnics, Kryen & Judd, McGraw Hill, 1957.

Discussion of
"GENERAL ASPECTS OF CEMENT GROUTING OF ROCK"

by V. L. Minear
(Proc. Paper 1145)

JUDSON P. ELSTON,* M. ASCE.—It is indeed a pleasure to see a paper which deals with the "mysteries" of grouting practices in as straight foreward a manner and as thoroughly and completely as Mr. Minear's. Mr. Minear speaks of "certain highly controversial practices upon which there is little unanimity of opinion." One wonders if these practices haven't developed because of the fact that every foundation material is constituted a little bit differently and subsequently requires variation in foundation treatment and procedures from job to job to insure an adequate foundation. In the study of geology there is found an infinite number of variations in structure and material of sediments and in sedimentary, metamorphic and igneous rock types in the original deposited undisturbed state. Engineers are often prone to forget the changes, past and those now going on, taking place in rock; not only dynamomorphic changes but changes attributed to the action of underground water both physical and chemical; at least until they are face to face with them during construction.

Certainly, as there are many discontinuities and unconformities—and actual unknowns in geology—so there are many unknowns in the treatment and correction of these weaknesses in foundations. Is not Mr. Minear implying that there are as many controversial practices as there are weaknesses in different rock types to be treated? If so, the writer certainly agrees with him. In fact, one wonders if this phase of engineering work is not still based as much if not more on individual judgment and ability as on past experiences Because an engineer has worked with or been "exposed" to a granite or sandstone or basalt does not always mean he is in a position to arbitrarily establish water-cement ratios, spacings or pressures before beginning the next job on a similar type rock.

Unless there are extremely weak or large water bearing seams, generally horizontal and generally in sedimentary rock, the writer has never been in favor of using the method of grouting, i.e. "zones of depth." Admittedly, if a weak layer or open water course exists at some depth below final rock grade for the structure, and it is found impossible to drill beyond this depth or, if drilled, to use packers beyond this layer or seam without first grouting it off, then zone grouting may become necessary. In effect, the end result is stage drilling and grouting requiring more drill and grout setups, more time to complete the work and often times a costlier operation. Don't misunderstand this premise. There is no argument with stage drilling and grouting when the foundation material is so poor that no other course of action is open. One thought that sometimes occurs under such conditions is if it wouldn't have been better to have considered excavating, mining or reinforcing the material

* Asst. to Constr. Mgr., Power Area, Uhl, Hall & Rich. Engrs., St. Lawrence Power Project, Massena, N. Y.

with concrete pillars and/or redesign or relocation of the structure. In the writer's experience (and here one is speaking of deep holes along a line to establish a barrier to water flows) there has been a strong inclination toward the practice of drilling a hole to full depth in one operation and then grouting it with packers (stops, plugs or whatever else they may be called) starting from the bottom of the hole upward in as many stages or packer settings as may be necessary to secure a satisfactory job. The contractor spends less time and money on the hole, the engineer has more freedom (range of pressures and water-cement ratios) in the treatment of the hole, and the operation is completed in the shortest time possible. There is a job on the St. Lawrence where the thinking is toward a combination of stage drilling and grouting and packer grouting to establish a grout curtain. Following which, the plan would be to drill and grout the somewhat porous and broken near surface foundation rock to whatever close spacings are necessary and complete this work before going deeper. From about 30 feet in depth to about 120 feet in depth (going back through the shallow grouted holes), deeper holes would be drilled to full depth at 80-foot centers, then grouted using the "closure" or split-spacing method to determine closer spacings and shallower depths. All holes below the 30-foot depth would be drilled to full depth in one operation. All grouting in these holes would be done with packers.

Mr. Minear's treatment of the subject of whether to do foundation drilling and grouting by "force account" or cost-plus, or whether to do the work by contract is interesting. The writer does not entirely agree with the statement that hired labor permits flexibility in field operations. On one job done for a municipality in the west, approximately 60 percent of the work was done under contract prices, and 40 percent was done by cost-plus. This with the same contractor, equipment, materials and labor throughout. Complete control and so-called flexibility of operations was maintained under the cost-plus feature; however, the incentive, initiative, the push of the contractor's people was not there. The results of both practices in quality of work were identical, but the costs, to the municipality, of the cost-plus work were higher and the time of completion on a hole and footage basis was longer. One objection to the work being done by the general or prime contractor is that, not owning the equipment or carrying the qualified men on his payroll, the quality of the work is lowered or a disproportionately higher inspection force is required to secure the desired end results. Again the writer is personally in favor of the experienced independent drilling and grouting contractor specialized in this work. He is able to move on a moment's notice, has a highly qualified nucleus of men, experience, and the knowledge of what he is able to do with various types of equipment. With such a contractor, of which there are many, the job can be done as quickly and efficiently as possible with only a bare minimum of inspection force.

Again referring to grouting of certain partial depths called "zones" it has been considered good practice to permit the driller to complete drilling of a hole to full depth providing he was able to do so without "burning in the bit" or losing the hole from "caving." In other words, the mere losing or making of water in a hole being drilled was not in itself considered justification to stop drilling and start grouting. On the other hand exploration, indicating a prevailing zone in which all water is lost or from which flows under pressure can be expected, may well lead to a pattern of holes drilled no deeper than the porous zone and then grouted simultaneously or consecutively. It is easy to think of as many different subterranean conditions requiring different

approaches toward a successful solution as there are words on this page. Preference has been to have the hole completed at one time whenever possible to drill it to full depth, keep it open, and insert and set a packer wherever desired.

Mr. Minear states that the problem of consistency of grout is largely academic. One certainly does agree that from a practical point of view it resolves itself into a question of whether something or nothing will be injected. A prime example was Hoover Dam in which grouts of 5:1 water-cement ratios or less (and generally less) were tried in the foundation during the construction stage. After the reservoir was filled, uplift pressures increased and large flows of water were had from the foundation drains. The only solution in order to seal off the rock and reduce uplift was to go to thin grouts on the order of 12:1 to 20:1 and to high pressures. In fact, some holes took thousands of sacks in this consistency range and yet they would seal off under an 8:1 and still release flows of water when uncapped. It is doubted if there are any outright advocates of thin grout or thick grout, although one meets some engineers who take a very dim view of grout consistencies greater than 5:1. Again these opinions so often are traced back to specific experiences and background. For instance, it may be difficult to convince an engineer from the Tennessee Valley Authority that a granite or granodiorite requires grouting at all if it will not accept a 4:1 grout or thicker. On the other hand, what would be their thinking on a hole that sealed off on a 5:1 after 200 sacks and yet made water at the rate of 100 gallons per minute when the valve was removed two days later?

In conclusion, the writer wishes to take the opportunity of congratulating Mr. Minear on this very excellent presentation of a somewhat controversial subject. Grouting will probably always remain a difficult subject to put into the printed word. Surprisingly, when engineers and contractors of varied background come together personally to discuss the subject and problems and differences of thinking on grouting, the controversy and argument seem to melt away to minor disagreements as to shade or meaning of words. This was amazingly demonstrated in October, 1956 at a meeting of the Committee on Cement Grouting of ASCE in Pittsburgh. Represented were engineers and geologists of Canada, as well as the Tennessee Valley Authority, the Corps of Engineers, the Bureau of Reclamation, private engineering firms, and contractors. Once knowing the others' background and the specific foundation problem or problems each had faced and solved, agreement was practically unanimous as to the individual successful procedure or technique which had been employed.

GLEBE A. KRAVETZ,[1] J.M. ASCE.—While the different aspects of cement grouting presented by the author are of interest, it is believed that pressure applied in grout injection and water pressure tests are of special importance. Further, it is felt that practices in foundation grouting are closely linked to the progress of grouting techniques. It is therefore, the intent of this discussion to comment on these specific points.

Pressure Applied

It is generally accepted that Prof. Maurice Lugeon established, in the early

1. Soil and Foundation Engr., Tippetts-Abbett-McCarthy-Stratton, Engrs., New York, N. Y.

thirties, the basic of modern grouting techniques. In his fundamental book[1] he recommends the use of grout pumps giving pressures of more than 50 kg/cm^2 (700 psi)—then a novelty—specifying that "it is only in the upper zones of horizontally stratified rock that such high pressures should not be applied for danger of upheaval, but for any other type of rock there is nothing to fear." In Europe, this advice has been followed for the last quarter of a century. Concurrently a safe rule of thumb was established to be used whenever rock conditions are not clearly definied: maximum pressure should not exceed 1 kg/cm^2 per meter of depth.*

Data illustrating the use of such pressures is not always readily available in the United States. However, a recent description[2] of the foundation grouting at the Santa Giustina Dam is typical. Consolidation grouting at a pressure of over 70 atmospheres** was carried at the abuttments through holes 43 meters deep in an average. The rock was a dolomite horizontally stratified. The following Table compares values of the maximum pressure used with pressures allowed by the American and European rules of thumb.

Pressure Allowed by Rules of Thumb

Depth of Holes	USA (1 psi/ft)	Europe 1 $kg/cm^2/m$)	Pressure Used
143 ft.	143 psi	620 psi	over 1028 psi

That the rule of thumb used in the United States is overly conservative has been recognized from the very time it was established. Much higher pressures have been used repeatedly, as can be seen from Table 1 of Mr. Minear's paper. Coefficients to increase the ratio of pressure to depth of rock have been proposed also.[3]

Pressure, however, is too important a factor to be merely determined by rules of thumb or by practical experience, no matter how successful. In the last few years, very important and successful studies have been made on the behavior of rock in tunnel and shaft work:[4] among others, tests have been run to measure the strains in tunnel linings during and after injection and determine the stresses before and after grouting.[5] It is strongly believed that similar studies could be planned and carried out to determine the behavior of foundation rock during grouting operations. Establishment of testing methods and procedures would not be easily accomplished (one has in mind, for instance, the installation of strain gauges in drill holes), and the interpretation of the results may be rather intricate, but the invaluable information which could be so gathered should justify the task.

Water Pressure Tests

The Bureau of Reclamation has two methods for water pressure testing.[6] In method #1 a single packer is used: successive zones are tested as the hole is gradually deepened. In method #2 a double packer is used: after the hole has been drilled to full depth, tests are started at the bottom and the double packer is gradually raised until the entire hole is tested. While detailed formulae to determine a coefficient of permeability K of the rock are given in the USBR "Earth Manual," no recommendations are made as to the pressures to be applied or to the length of the zone to be tested. In practice, however, it

* 1 kg/cm^2/meter = 4.33 psi/foot
** 1 atm = 1.033 kg/cm^2 = 14.69 psi

is often found that the values of K, as obtained from water pressure tests, are not truly independent from the pressure and length of zone. Values for these two factors should be kept constant throughout a job in order to obtain comparative results.

Prof. Lugeon(1) proposes a method of testing similar to method #1. In addition, however, he recommends (a) a constant length of zone: 5 meters; (b) pressures up to 10 kg/cm^2; (c) the absorption to be determined from tests in which a pressure of 10kg/cm^2 is applied for 10 minutes and expressed in liter/meter/minute. This unit is often called the "lugeon," and is found to be equivalent to a coefficient of permeability K varying between 1 and 2.10^{-7} meter/second.(7) He recommends also to stop the drill holes when the absorption is less than 1 lugeon for dams over 30 meters high and 3 lugeons for smaller dams.

It is interesting to note that in the three methods of testing described, each hole is tested during drilling or immediately after completion. In Europe, the practice is to make water pressure tests not only during preliminary subsurface exploration, but during the grouting program as well. Each zone in a hole is tested immediately before its grouting and the grout consistency is determined according to the test results.

Usefulness of the water pressure tests cannot be overemphasized. However, it is only by proper methods and careful interpretation of the results that reliable data can be obtained.

Practice

In a recent book(7) Prof. Cambefort, discussing practices in pressure grouting, advocates the need for firms specialized in grouting. He points out that while these firms are common in Europe, in the United States general contractors handle this type of work. But, adds Prof. Cambefort their personnel is not sufficiently skilled and their technique does not reach European levels, as illustrated by the following examples:

"It was necessary—a thing which never happened in Europe—in 1939-40, after the Boulder Dam reservoir was filled, to deepen the grout curtain because of an insufficient exploratory program.

To our knowledge, Americans have not succeeded yet in satisfactorily sealing alluvial strata by grouting methods, while many years ago the deep cutoff of the upstream cofferdam of Genissiat (dam) was so sealed, not to speak of progress made since then: Ottmarsheim, Fessenheim, Serre-Poncon (dams), etc."

The specialized firms referred to, usually have an engineering section with a staff of geologists and foundation engineers, as well as a soil and material laboratory. They very often carry research work and sometimes manufacture their own drilling and grouting equipment. Engineering firms or government agencies in charge of design and supervision of hydroelectric work frequently retain a specialized firm to study the geological conditions at a dam site, evaluate the foundation treatment and prepare drawings and specifications for the drilling and grouting contract.

The conditions existing in Europe are believed to be significant: profit is still the prime incentive for progress. A specialized contractor who can bid on separate contracts covering the drilling and grouting is more interested in new methods and techniques than a general contractor for whom, in a multimillion job, grouting is a minor item, or than an Engineer assigned to the job with a hired crew.

REFERENCES

1. "Barrages et Geologie" by M. Lugeon. Paris, France.
2. "Arch Dams: Santa Giustina Single-Curvature Arch Dam" by C. Marcello. ASCE Proceedings No. 992.
3. "Notes on the Theory and Practice of Foundation Grouting" by V. L. Minear. Journal, A.C.I. April 1947.
4. "Present Trends in the Design of Pressure Tunnels and Shaft for Underground Hydroelectric Power Stations" by Ch. Jaeger. Proceedings I.C.E. March 1955, and Bibliography included.
5. "Measurement of Moduli of Elasticity and their Application to the Design of Pressure Shafts" by P. Bernard. Proceedings Third Int. Conf. Soil Mech. & Foundation Engineering. Switzerland 1953.
6. "Earth Manual" U. S. Dept. of Interior, Bureau of Reclamation. Denver, Colorado. Feb. 1952.
7. "Forages et Sondages" by H. Cambefort. Paris, France, 1955.

Discussion of
"MOISTURE CONDITIONS UNDER FLEXIBLE AIRFIELD PAVEMENTS"

by J. F. Redus
(Proc. Paper 1159)

JOHN W. GUINNEE* and CHARLES E. THOMAS.**—The writers wish to express their appreciation for Mr. Redus' fine report on a problem which has been the subject of much theoretical consideration but about which there has been little information published concerning extended field studies. It has been their concern for some years, to accumulate and analyze similar data in regard to subgrade moisture conditions under major highways in Missouri, constructed of concrete pavements on stone bases. It is with pleasure, that the writers would like to sustain some of the author's conclusions wherein the two studies are comparable.

Some of the major differences which exist are that the writers are considering 8" by 24' concrete pavements built on 4" by 26' rolled stone bases, while the author considers flexible pavements approximately 150' wide with wearing courses varying from 1 1/2" to 3" in thickness on multiple type bases varying from 8 1/2" to 14" in thickness. The writers' data include approximately the same number of locations at grade, in cut, or on fill while it must be assumed that most of the author's study sites were approximately level (at grade).

Normally, the general climatic conditions of Missouri would most nearly approach those at Stewart Field except that Missouri has experienced a drouth thru the years 1948-1955 for which data have been gathered and analyzed. The effect of these drouth years on subgrade moistures has yet to be determined, but it is hoped that continuing studies will throw additional light on the subject. It can only be conjectured that the drouth may have provided a retarding effect on the rise of the general moisture curve but at the same time, provided a larger spread in moisture variations from minimum average to maximum average, because of the lower minimums involved.

Using average yearly subgrade moisture contents divided by the Plastic Limit and plotted against the total yearly rainfall, Figure 1. shows the same general relationship as illustrated by the author. Figures 2. and 3. are reconstructed graphs taken from data presented in a recent paper.(1) These graphs sustain the author's conclusion that moisture generally reaches a plateau within the first two years after construction. The prolonged drouth of 1952, 1953 and 1954 served to lower this plateau slightly. Other data from an unpublished report(2) show that during those drouth years the average subgrade moistures varied as much as 15 percentage points. This variation is probably due to the great desiccating effects of the drouth so that the changes in subgrade moisture closely followed the individual rainfall occurrences.

Other unpublished data(3) collected from 1937 to 1947 concern subgrade moistures under various experimental types of base construction for a

* Senior Engr., Missouri State Highway Dept., Jefferson City, Mo.
** Materials Engr., Missouri State Highway Dept., Jefferson City, Mo.

bituminous armor coat surfacing on Route 100, Franklin County, Missouri. Figure 4. shows the average rise in moisture at two locations; 2 ft. from the edge and 8 ft. from the edge, at three depths in the subgrade. As can be readily seen there was a rapid rise in the moisture during the first month of cover at locations 2 ft. from the edge, which seemed to be essentially completed in one year. On the other hand, locations 8 ft. from the edge took nearly seven years to rise to approximately the same level. There have been many discussions of the reasons why such a slow rise should have occurred. At the present time it is the consensus that the combination of a well maintained surface armor coat, high roadway crown, deep (2 1/2' below edge) clean ditches, and high count—medium weight traffic among other environmental conditions, accounted for this slow moisture rise.

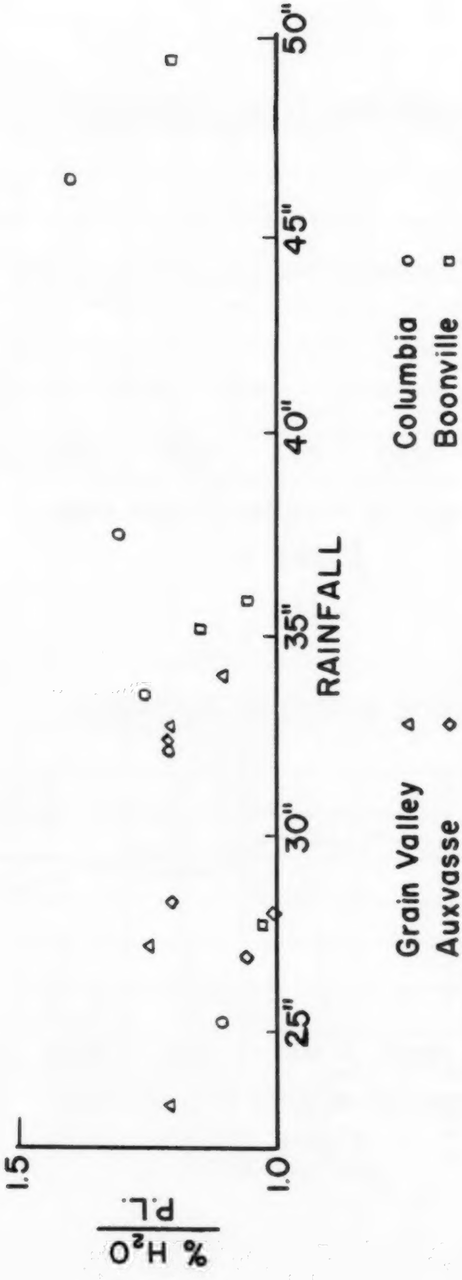
It is interesting to note that within the writers' observations the use of base course construction under concrete pavements in Missouri has proven efficacious in the prevention of pumping (i.e. the original purpose of the design). Up to the present time locations which show high subgrade moisture contents have not evidenced severe pavement distress and collaterally severe pavement distress, where existent, cannot be attributed solely to high subgrade moistures.

A summarization of the writers' observations might be;

1. Moisture conditions in the top levels of base and subgrade show more quickly the effects of rainfall and consequently show greater variations.
2. Deeper subgrade levels respond more slowly to rainfall variations and therefore more generally reflect the average yearly rainfall.

REFERENCES

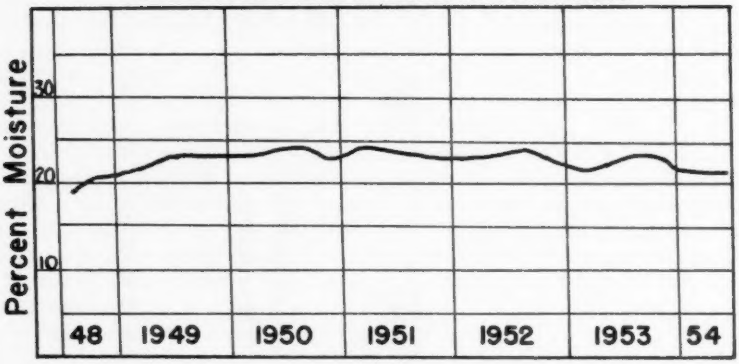
1. Guinnee, John W. and Thomas, Charles E. "Subgrade Moisture Conditions under an Experimental Pavement" Highway Research Board, Bulletin 111, 1955.
2. Guinnee & Thomas "Subgrade Sampling through Pavement Thickness Core Holes" Progress Report, Missouri State Highway Department.
3. Lancaster, C. M., Senior Engineer, Missouri State Highway Department "Report on Subgrade Moistures, Route 100 Franklin County."



IN-PLACE MOISTURE CONTENT / PLASTIC LIMIT vs. ANNUAL RAINFALL

Figure I

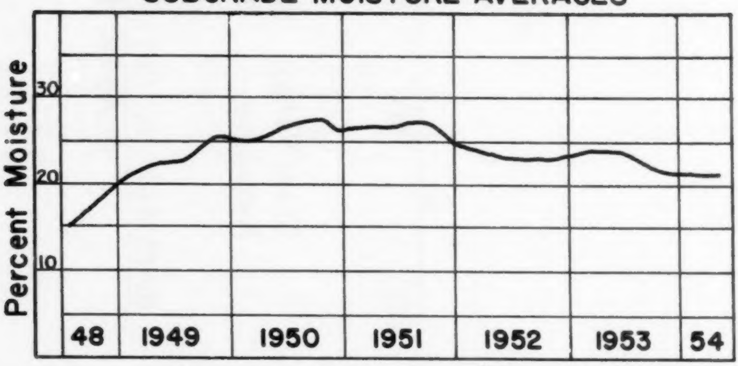
SUBGRADE MOISTURE AVERAGES



Average of all sites 3' from edge

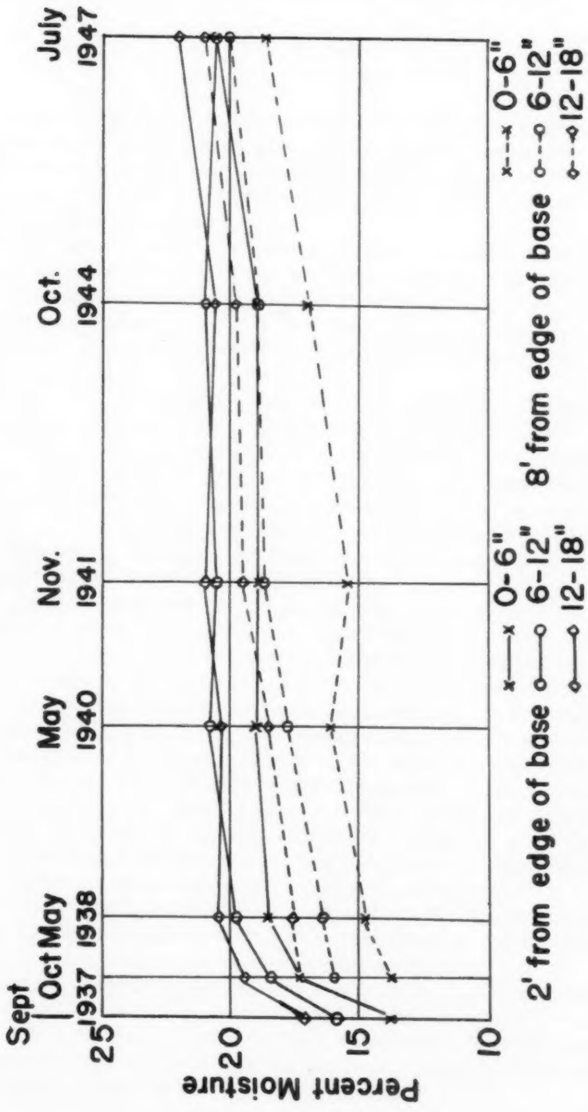
Figure 2

SUBGRADE MOISTURE AVERAGES



Average of all sites 9' from edge

Figure 3



AVERAGE SUBGRADE MOISTURE
Rt. 100, Franklin Co.

Figure 4

Discussion of
"FRICTIONAL RESISTANCE OF STEEL H-PILING IN CLAY"

by Eben Vey
(Proc. Paper 1160)

LOUIS F. MENARD,[1] J.M. ASCE.—This valuable paper presents interesting data on the effects of H-pile driving on soil properties and on a comparison of the soil resistance in the field with the shear strength measured in the laboratory. Though most of the conclusions seem correct to the writer, two of them appear questionable.

1. The author, after comparison of the unconfined compressive strength before and after driving of the H-piles, concludes that the clay does not suffer any great sudden loss in strength due to the pile driving. The writer has carried out, in Chicago, a series of tests to measure the effects of H-pile driving on the soil with a new apparatus called a pressiometer and has reached different conclusions.

2. Mr. Vey states that there is a good correlation between the shearing strength of the soil around the piles in the field and that found in the laboratory; the writer believes that the H-pile driving and sampling disturbance caused equivalent remolding of the Chicago clays; this explains how a reasonable agreement may be found between the shear strength of the soil in the field and in the laboratory; in spite of its shortcomings, the unconfined compression test is therefore capable of predicting the shear strength of clays of medium sensitivity after H-pile driving.

A pressiometer test is run as follows: a cylindrical measuring cell is lowered to the desired depth in a bored hole of the same diameter. The wall of the cell is made of rubber membrane and can expand laterally. It strains the soil radially around it when water under pressure is supplied inside the cell. The increase in diameter of the bored hole is proportional to the increase in volume of the cell and consequently to the quantity of water supplied. The increase in pressure exerted on the soils are recorded by a Bourdon Gauge. To get a nearly cylindrical stress distribution in the vicinity of the measuring cell, two dummy cells are fitted above and below the first one and the same pressure on the soil, as shown on Fig. 1.

The diameter increase of the hole is plotted versus the pressure. The soil properties can be evaluated in a manner analogous to the interpretation of load tests. The apparatus was designed and the theory developed by the writer at the Ecole Nationale des Ponts et Chaussees in Paris, and was further developed and evaluated at the University of Illinois.

This equipment, which is very cheap and time-saving, has been found to give results in agreement with those of routine or more conventional tests. It has the advantage of permitting a determination of the elastic properties of the soil without the necessity for extracting samples with accompanying likelihood of disturbance.

In June 1956 a series of tests was completed on the site of the new Inland

1. Research Asst. in Civ. Eng., Univ. of Illinois, Urbana, Ill.

Steel Building in downtown Chicago. The tests were made 33 ft. below the street level after excavation had exposed the tops of the H-piles driven in January, 1956, by means of a follower. Thirty pressiometer tests were run at different depths and different distance from a single pile or from a cluster of piles as shown on Fig. 2.

On Fig. 3 are plotted several pressiometer curves showing the increase in diameter of the hole versus the pressure applied on the wall of the hole. When the pressure is progressively increased the corresponding increases in diameter are proportional to the pressure during the elastic phase, then become larger and larger, and finally increase without limit for the ultimate value of the pressure.

Table 1 shows a comparison between the value of the modulus of elasticity, shear strength and ultimate pressure yielded by the pressiometer tests and the corresponding values given by the unconfined compression tests.

According to the pressiometer tests the driving of the cluster of the H-piles had a remolding effect in Chicago clays. The decrease in shearing strength reached a mean value of 40% and the modulus of elasticity decreased from 30 to 6 tons per sq. ft.

On the other hand, the results of the unconfined compressive tests showed no variation in shear strength or modulus of elasticity in relation with the distance to the piles or to the cluster of piles. The remolding due to the sampling counteracted and even completely obscured the effects of pile driving.

It may be noticed that in the neighborhood of the pile (or the cluster of piles) the pressiometer tests and the laboratory tests gave approximately the same value for the modulus of elasticity and the shearing strength.

Though the results justify a more complete investigation, it is apparent that reliable tests in the field are necessary to measure the remolding effects of pile driving.

The tests were made through the courtesy of Skidmore, Owings and Merrill, architects and Engineers for the Inland Steel Building, with the assistance of Mr. Yves H. Lacroix, Research Assistant in Civil Engineering, University of Illinois.

TABLE 1

Series	Distance from pile (or cluster)	Modulus of elasticity U.C.I.	Modulus of elasticity Pressiometer	Shearing Strength U.C.I.	Shearing Strength Pressiometer	Ultimate pressure
N.1 (1 pile)	4"	-	6	-	.25	1.2
	1'6"	-	27	-	.45	2.7
	2'6"	-	41	-	.45	3.2
N.2 (one pile)	6"	7.5*	6	.40*	.34	1.7
	1'0"	-	13	-	.46	2.2
	3'0"	-	29	-	.46	3.0
	5'6"	11.0*	28	1.52*	.50	3.2
N.3 (a cluster of 8 H-piles)	inside of the cluster	6**	6	.38**	.46	1.8
	1'6"	-	25	-	.55	2.8
	4'6"	-	23	-	.60	2.85
	7'6"	6**	28	.38**	.70	3.5

Note:
* Shelby tube samples pushed with a hammer.
** Shelby tube samples driven into the soil.

Each of the values above are the mean values of three or four tests.

All values in tons per sq ft

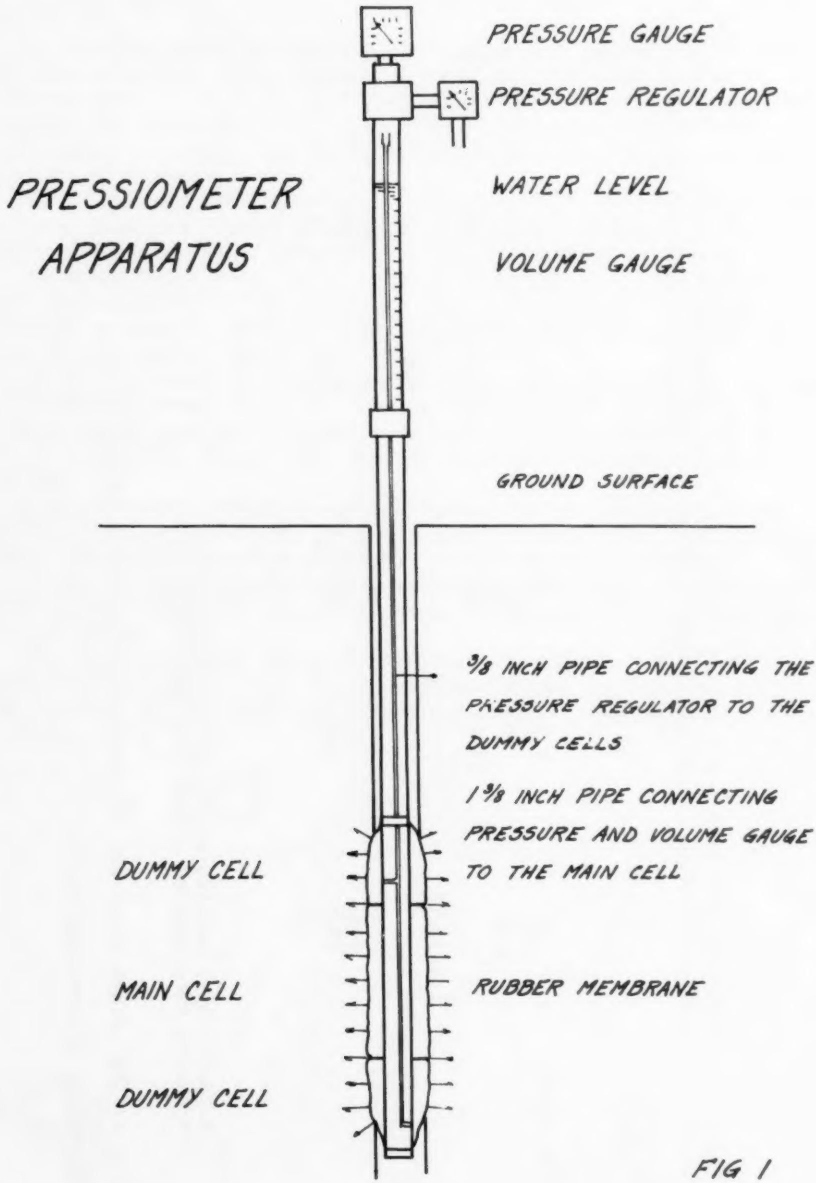
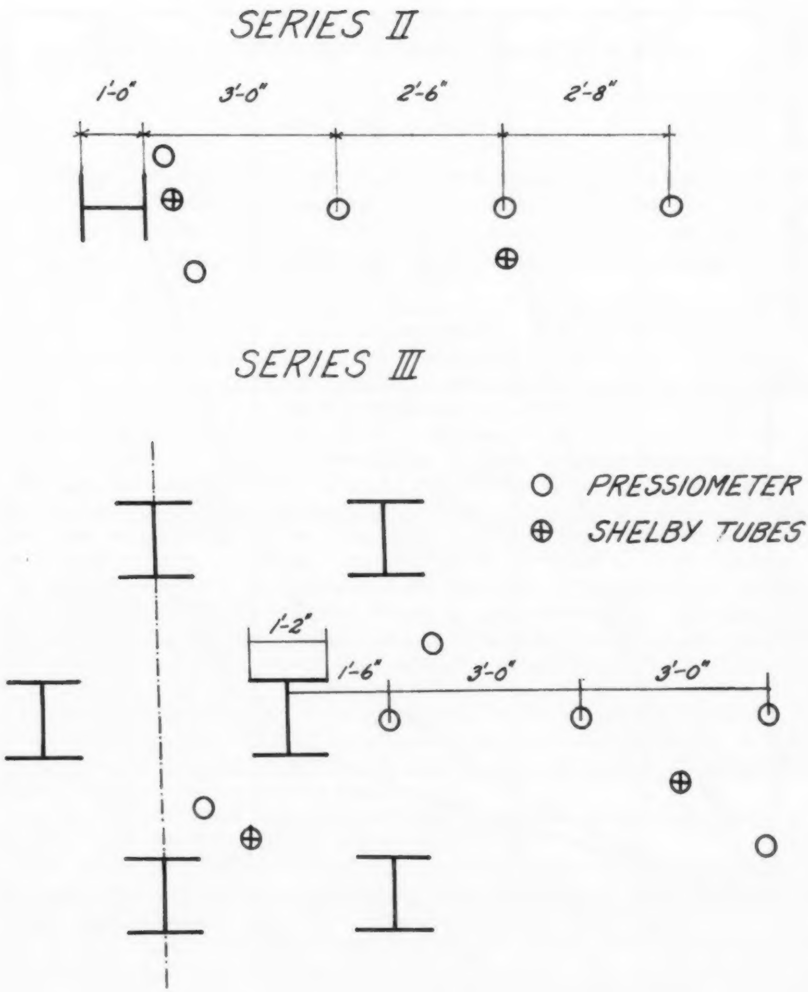
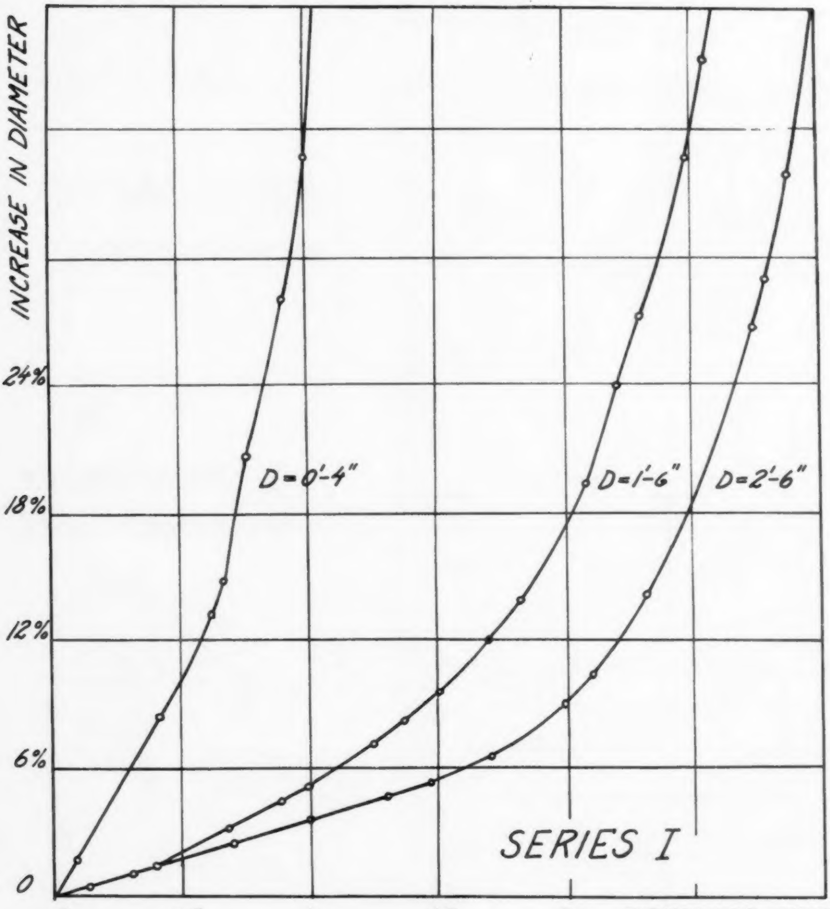

PRESSIOMETER APPARATUS
PRESSURE GAUGE
PRESSURE REGULATOR
WATER LEVEL
VOLUME GAUGE
GROUND SURFACE
3/8 INCH PIPE CONNECTING THE PRESSURE REGULATOR TO THE DUMMY CELLS
1 3/8 INCH PIPE CONNECTING PRESSURE AND VOLUME GAUGE TO THE MAIN CELL
DUMMY CELL
MAIN CELL
RUBBER MEMBRANE
DUMMY CELL

FIG 1



LOCATION OF BORINGS

FIG 2



PRESSIOMETER DIAGRAMS FOR VARYING DISTANCE FROM PILE

FIG 3

Discussion of
"RELATIVE DENSITY AND SHEAR STRENGTHS OF SAND"

by T. H. Wu
(Proc. Paper 1161)

ALFRED A. SEYMOUR-JONES,* J.M. ASCE.—This paper by Mr. Wu has contributed to a better understanding of the characteristics that determine the behavior of noncohesive soils. Information of this nature provides a basis for estimating the behavior of these soils. He is to be commended for this work as too little is known in this field at present.

In this paper there are several statements that this writer would like to comment on. The mention of the methods used for determining the maximum and minimum voids ratio are of prime importance to the writer. For the important concept of relative density to have meaning, the numerous proposed methods used to determine the maximum and minimum densities should be modified so that discrepancies between their results are eliminated. Only then will a specific relative density imply the same meaning to all people.

Because the maximum and minimum relative densities or void ratios are meant to correspond to the densest and loosest conditions of a soil in the natural state, the methods of determining the maximum and minimum densities should be devised on this basis. The method Mr. Wu used to obtain the minimum density appears to be reasonable and adequate. The method for obtaining the maximum density, however, seems to produce densities less than that obtainable for the soil in the natural state. The writer feels that the use of vibratory compaction or the Modified AASHO compaction method would produce more realistic values for maximum density of noncohesive soils.

The effect of segregation, noted by Mr. Wu, on the values for minimum density are important. This effect indicates that the supporting value of segregated soils in the natural state have a better bearing value than would be indicated by their relative density alone.

The author noted that in natural deposits, soils with the greater mean diameter were found to have the higher void ratios.

This condition suggests that the soils with the larger mean diameter have, at a lower relative density, a supporting value equal to that of the soils with a smaller mean diameter at a higher relative density. This idea, carried one step further, implies that for a constant relative density, the supporting value of soils increases with the mean diameter. The latter is in agreement with a rating table for granular soils published by Professor Burmister of Columbia University.[1] This table rates the relative supporting value of granular soils on the basis of relative density.

It was noted that the consolidation curves in Figure 10 of Mr. Wu's article tend to support this idea. The soils, all placed initially at their minimum density, showed that the compressibility increased with decreasing mean

---

* Soils Engr., Howard, Needles, Tammen & Bergendoff, New York, N. Y.

1. Burmister, D. M. "The Importance and Practical Use of Relative Density in Soil Mechanics," Proceedings of the American Society for Testing Materials, Volume 48, 1948.

diameter exhibit less consolidation under the same load and therefore have better supporting value with regard to settlement.

Undoubtedly there are other soil properties such as gradation and grain shape that will affect the relative supporting value of granular soils besides their relative density and mean particle size which are discussed here.

From Mr. Wu's studies it appears that segregation has the greatest effect on density in the coarse grained soils. Another factor that could control the relative change in density of a soil due to segregation is the range of grain sizes the soil encompasses. That is, for two soils having the same mean diameter, the soil with the larger range of grain sizes would have the greatest density change from segregation.

The effect of segregation has important meaning especially where density control is applied to natural or constructed deposits of granular materials. Under the same compaction effort a segregated soil will have a higher void ratio and lower density than the same soil with no segregation. Therefore, field compaction requirements should take into account whether the soil is segregated or not. Laboratory compaction tests, which usually are the basis for such control, should be modified accordingly, otherwise unnecessary field compaction effort will be needed to meet the requirements.

The effect of relative density on the shear strength of granular soils is important as Mr. Wu has shown. The writer performed a similar series of tests on a sand in connection with a slide investigation. Professor Burmister has published the results of similar tests.[1]

The writer's tests were limited to one sand. Increase in the angle of internal friction with density was noted which is in general agreement with the tests performed by Mr. Wu. Professor Burmister tested several granular soils having a wide range of grain sizes and grain size distributions. The results of his tests indicate that conditions besides relative density affect the angle of internal friction of granular soils, although relative density is the most important. His tests showed that the degree of control of these other conditions is greatest at low densities, and decreases with increased density.

Professor Burmister mentions that the grain size distribution appears to be an important condition. It is quite possible that grain shape is another.

The grain size curves for the three sands tested by Mr. Wu had approximately the same shape. Since these soils were obtained from the same deposit, it is most likely they have the same grain shape. Probably the only major difference between these samples is the mean grain diameter. The general similarity of the three samples tested by Mr. Wu could account for the close agreement obtained of ϕ vs. relative density.

It would be difficult to directly compare the results of the ϕ—density tests performed by Messrs. Wu, Burmister and the writer. The difficulty arises because Mr. Wu has used a procedure of determining 100 per cent relative density that apparently produces a lower density than the vibrational compaction used by Professor Burmister or the Modified AASHO procedure employed by the writer.

Although granular soils generally are better actors than cohesive soils, they still are the causes of structural problems. Only through a better understanding of the relation among their properties, environment and reaction to loads can these problems be minimized or eliminated. Therefore, further basic research similar to that performed by Mr. Wu is required.

BYRON J. PRUGH,* A.M. ASCE.—The data presented by the author extends further the limited published information on soil properties in situ. As the author pointed out, the process of sedimentation produces soils whose relative densities are a function of their particle size. Emphasis however has not been given to a more important factor affecting the relationship between relative density and mean or peak particle size.

This important factor is the type of deposition under which the sedimentation occurred such as alluvial, lacustrine, glacial, aeolian etc. This variation, mentioned in the author's paper, is shown graphicly in Figure 1 of this discussion where the alluvial sand of the Lower Mississippi River is compared with the two glacial soils shown in the author's Figures 8 and 9.

It should be particularly noted that the relative density of the alluvial soil increases with increasing grain size, yet for the glacial soils it decreases. In a similar manner, other types of depositations will produce other curves. If a sufficient number of accurate investigations, similar to that made by the author, are performed on soils deposited under different conditions, a practical chart, consisting of curves for each type of depositing, could be compiled. This chart should be of value for any site investigation.

The use of such a chart or graph would be comparatively simple. One of the first steps in a soil study is an investigation of the geology of the area which would reveal the approximate type of geological deposits in the area.

The importance of Relative Density, especially of cohesionless soils, has been relatively understressed as a design factor, yet it has a decided influence on both bearing values and permeability. While relative density is easily understood, little has been published on the actual values as found in the ground. The author's work adds to the fund of soil knowledge and of a soil not usually encountered.

---

* Soils Research Director, Moretrench Corp., Rockaway, N. J.

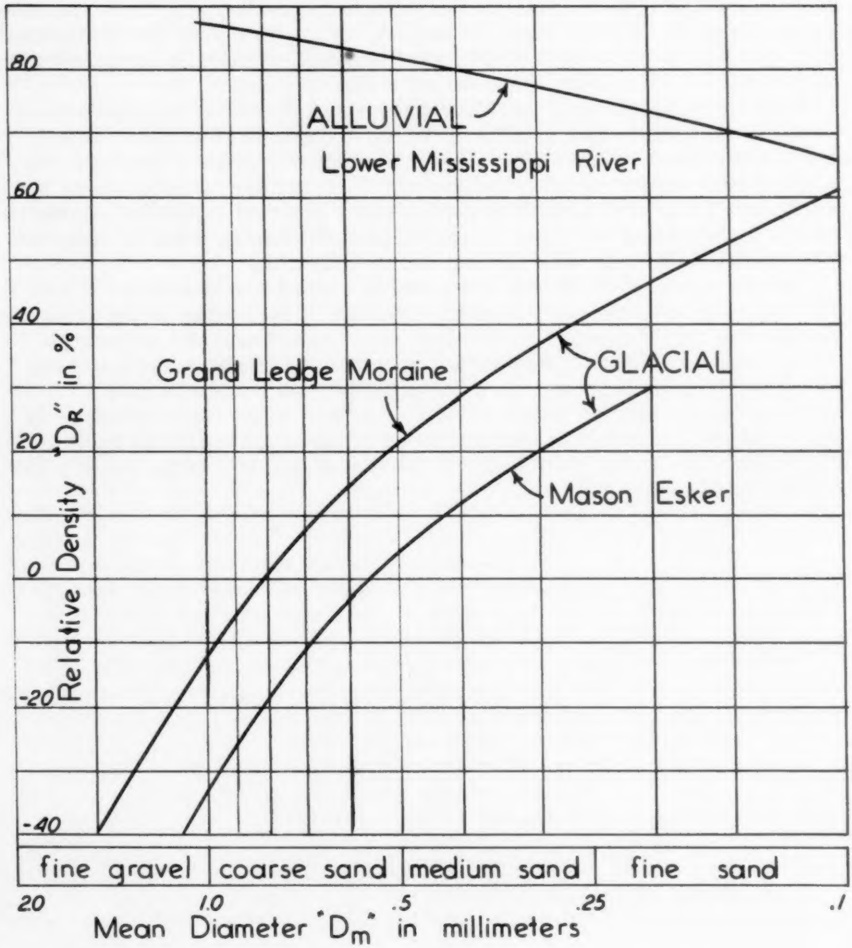

80
60
40
20
0
-20
-40
ALLUVIAL
Lower Mississippi River
GLACIAL
Grand Ledge Moraine
Mason Esker
Relative Density "D_R" in %
fine gravel
coarse sand
medium sand
fine sand
20
1.0
.5
.25
.1
Mean Diameter "D_m" in millimeters

Figure 1.

Discussion of "MISSISSIPPI VALLEY GEOLOGY—ITS ENGINEERING SIGNIFICANCE"

by C. R. Kolb and W. G. Shockley
(Proc. Paper 1289)

SHU-T'IEN LI,[1] M. ASCE.—This paper on the engineering significance of the lower Mississippi valley geology, as the authors have covered at length, is a valuable contribution toward clarifying the various environments of alluvial deposition and the resulting soil formations in relation to their associated engineering problems. Though the title of the paper does not expressly indicate, its contents have referred to and contexually implied, the whole lower Mississippi alluvial valley.

Between the valley walls and above the Pleistocene substratum or Pleistocene valley floor, the recent substratum and topstratum have been formed after the Mississippi River entrenched its valley system during the Later Wisconsin glacial stage, the last of the five Pleistocene glacial stages of the accepted world-wide Quaternary cycles. During each stage, as ice accumulated and melted, sea level was lowered and later restored to near its present stand. Each entrenchment and later filling of the valley system was, in each event, in an effort to adjust itself to the changed base level thereby imposed on it. The last valley filling began when sea level started its rise with the melting of the Later Wisconsin glaciers some 25,000 to 30,000 years ago, and the sea level reached its present stand some 5,000 years ago.

Lower Region of the Lower Alluvial Valley

While the alluvial valley of the lower Mississippi River forms, as the authors state, an elongate lowland extending from Cairo, Illinois, to the Gulf of Mexico, the more complex and striking events have happened in the lower region of this alluvial valley. The lower region may be regarded as lying south of the latitude of Vicksburg, Mississippi, flanked on the east and west sides by Pleistocene uplands between which the Mississippi has shifted its meandering courses over the previously filled valley systems.

As the authors have treated in a masterly fashion the various environments of alluvial deposition and the resulting soil formations in the lower Mississippi alluvial valley, and even at more length in the extremely lower-left region of the Lake Pontchartrain area, the writer's discussion will be confined to the more striking aspects of the lower-right region from Vicksburg, Mississippi, down to the Gulf of Mexico, and from the former Teche Mississippi on the west to the Modern Mississippi on the east. This lower-right region has predominently been the shifting region of the lower meander belts.

1. Chf. Technical Writer, Palmer and Baker, Inc., Cons. Engrs., Mobile, Ala. Formerly Deputy Pres., Yellow River Comm., China.

Post Pleistocene Epoch Events of the Lower-Right Region

To comprehend the geological aspects of the lower-right region of the lower Mississippi alluvial valley, an account of the post Pleistocene-epoch events thereof seems necessary. The associated events may be conveniently accounted for in (A) the period of recent substratum formation during the pre-meander-belt-shifting stage, and (B) the period of recent topstratum formation during the meander-belt-shifting stages.

(A) Recent Substratum Formation during the Pre-Meander-Belt-Shifting Stage

a) Under the present substratum are the Pleistocene and older deposits in which the Mississippi River and tributary streams entrenched their valley system during the Later Wisconsin glacial stage to as deep as 450 feet at places below the present sea level. The then exposed surface of this valley system experienced a weathered zone.

b) This deep valley system was later filled with the coarse basal fraction of the alluvial sequence, consisting principally of sand and gravel in the lower substratum and abundant sand in the upper substratum—together forming the recent substratum. The coarser materials were deposited when the river had a steeper slope and was overloaded. The thickness of the substratum increases from north to south and decreases from the axis of the entrenched valley to vertually none in places near the Mississippi alluvial valley wall. Below the Upper Grand River the thickness of the substratum is generally in the neighborhood of 300 feet or more and increases to as much as 400 feet further south. South of the latitude of Chicot Pass, Louisiana, gravel even of pea size is scarce above the elevation of about -160 feet M.G.L.

c) After the former entrenched valley system had been filled by the coarser materials, the entire lower valley floor became a wide more-or-less graded downward sloping surface toward the Gulf. As the slope was still moderately steep and the river remained to be overloaded, the exhibition of a braided channel habit ensued. Materials reaching the Mississippi were swept seaward by the numerous shallow channels of the braided stream and were deposited over wide areas as the aggrading valley surface developed. As slope decreased, the deposited material became finer toward the surface of the recent substratum and in the upstream direction. The generally coarse basal portion of the alluvium, called the recent substratum, accumulated in this manner and has a comparatively flat top surface.

d) The comparatively flat top surface formed during the above braided stream stage extends almost from east valley wall to west valley wall. This marks the top of the recent substratum over which the meander belts have shifted during the last 3,000 or more years.

Substratum and Recent Substratum

In the last paragraph under "Introduction," the authors ascribe the sand and gravel deposits to the substratum and, in the first paragraph under "The Substratum," state: "The substratum consists of a wedge of coarse-grained material laid down during the earlier stages of the filling of the entrenched valley of the Mississippi River." This latter statement confines the

substratum to within the entrenched valley during the Later Wisconsin glacial stage. An examination of the substratum profile from the Mississippi River near Union to north of St. Martinville, Louisiana, made by the Southern Laboratories, Inc., for the projected Acadian Thruway, shows that an unbroken top surface of sand substratum exists throughout the entire trans-Atchafalya line. The top surface of this sand substratum lies generally between -90 feet and -120 feet M.G.L. Only near the east-side and west-side Pliestocene uplands, it rises to -30 feet M.G.L. As the last entrenched valley during the Later Wisconsin glacier stage could not be as wide as 64.4 miles from valley wall to valley wall, this unbroken sand substratum should have been laid down from time to time when the Mississippi had the braided-stream character after the filling of the last entrenched valley. The authors confirm this view under "Braided Stream Deposits" by stating: "The sands and gravels of the substratum undoubtedly were deposited by such streams."

It appears more distinctive to call the Pleistocene floor as the substratum and the entrenched-valley filling and the braided stream deposits as the recent substratum. It is on the top part of this recent substratum that most of the heavy constructions in the lower region of the lower Mississippi alluvial valley have been and will be founded.

While the authors' statement, "Although braided stream deposits cover large portions of the alluvial valley of the Mississippi they are usually so distant from the large streams that few major engineering projects have been concerned with them," may be true from Cairo, Illinois, down to Natchez, Mississippi, it does not appear so from Natchez down to the Gulf, because systematically projected borings referred to above indicate that the comparatively flat top of the recent substratum reaches on the west to the Teche-ridge and on the east to the present Mississippi course. And in this lower region of the alluvial valley, the ancient braided stream deposits are entirely buried underneath the meander belt deposits and backswamp deposits.

Of particular engineering significance for heavy construction is the existence or non-existence of coarse sand and gravel in the recent substratum in the lower alluvial valley. It was reported, in connection with the geological investigation of the Atchafalaya Basin made by the Waterways Experiment Station, that no such material was found in any of the bed material samples taken south of the Upper Grand River. This information at least indicates that the occurrence of gravel in the substratum south of the Upper Grand River is very rare, except at great depths. The log of a boring located nearly a mile south of Chicot Pass shows that pea gravel exists below -158 feet M.G.L. Of equal engineering significance is the fact that the age determination of the samples taken from the Atchafalaya Basin indicates that a considerable amount of compaction of the sediments, due to natural consolidation, has taken place since deposition. It is reasonable to expect that the older deposits corresponding to the different ages and stages, other things being equal, have become more compacted due to natural consolidation.

The Pleistocene substratum valley floor slopes rapidly upwards near east-side and west-side Pleistocene uplands. During the former Teche-Mississippi and the Modern-Mississippi meander belt formation stages, there were undoubtedly meander loops coming adjacent to the valley walls. Thus point bar or accretion deposits of sand ridges were formed on the insides of river bends. Later meandering migrations turned away from the valley walls, leaving these sand bar deposits outside of natural levee formations or artificial levee building in the case of the present Mississippi. This explains the

condition why the borings made by the Southern Laboratories, Inc., for the projected Acadian Thruway show that the recent substratum rises to -30 feet M.G.L. near the east-side and west-side Pleistocene uplands. It also confirms the statement under "Point Bar Deposits" made by the authors where they expressed: "In the southern portions, depth of sand ranges from 10 to 30 feet." Though these sand deposits may be contiguous with the sand substratum, they were not left by the braided stream but were deposited during the meander-belt-shifting stages. An extreme case is shown by the projected boring L-5, made in connection with the Geological Investigation of the Atchafalaya basin by the Waterways Experiment Station, which is located west of Donaldsonville, Louisiana. The log shows that brown clay with shell extends from the Lafourche ridge at +20 feet to -2 feet, then sand and gravel to -180 feet. Here again, the high-elevation of sand and gravel deposits must have their origin from point bar deposits of the Lafourche stage.

(B) Recent Topstratum Formation During the Meander-Belt-Shifting Stages

a) The recent topstratum was deposited over the recent substratum as the river gradually became graded to the stand in sea level and ultimately the graded condition extended up the main tributaries. As a result, sediments carried to the Mississippi by tributaries became increasingly finer, the coarse fraction of the load diminished, the tendency for the stream to be overloaded decreased, the larger portion of the sediments could be carried in suspension by the master stream, its braided channel was gradually replaced by a meandering one. This brought about a concentration of the flow into a single channel, and resulted in deeper scouring action, period flooding and systematic channel migration. The majority of the materials deposited, since the Mississippi became a meandering stream throughout the major portion of its lower valley, consists of clays, silts, and fine-grained sands which make up the recent topstratum.

b) The oldest meander-belt of the Mississippi that geologists have been able to detect authoritatively is the Maringonin-Mississippi. It started to develop approximately 3,000 years ago, traversing longitudinally beneath the present Atchafalaya Basin. About 2,000 years ago, the Mississippi abandoned the Maringonin meander course, leaving there an alluvial ridge, and commenced to develop the Teche-Mississippi meander belt near the foot of the western Pleistocene uplands. About 1,600 years ago, the Mississippi abandoned the Teche meander course, and started to develop the Lafourche meander-belt to the east of the Maringonin-Mississippi alluvial ridge. The Modern Mississippi, forming the most easterly meander belt, was first occupied approximately 800 years ago.

As pointed out by the authors, meander belt deposits are among the most important in the alluvial valley. The above account, based upon findings made by authoritative geologists, gives the different ages during which the different meander belt deposits that comprise (a) natural levee deposits, (b) point bar deposits, (c) abandoned channel deposits of "clay plugs," and (d) abandoned course deposits, were laid down over the recent substratum of the alluvial valley. All these groups of deposits have been well accounted for by the authors.

Geomorphologically, not all of the meander-belt ridges are discernible at the present. North of Donaldsonville, Louisiana, and for some distance south thereof, the Modern-Mississippi meander belt overlies the next older

Lafourche-Mississippi meander belt. It has been known as Modern Lafourche-Mississippi meander belt. The oldest meander belt, i.e. the Maringonin-Mississippi, has long been over-alluviated by the backswamp deposits of the Teche-stage, the Lafourche-stage, and the Atchafalaya-stage. Though a few low arcuate features having the dimensions of the Red and Mississippi River meander loops are discernible from aerial photographs, they cannot be followed as uninterrupted ridges. These are remnants of the Maringonin-alluvial ridge now largely buried beneath more recent deposits.

The Teche-ridge on the west and the Modern-Lafourche-ridge on the east form the basin boundaries of the growing Atchafalaya stage. These boundary ridges were formed mainly of bar accretions laid down within the then Mississippi meander-courses and were capped by the broad natural levees of the Mississippi and have since been capped by narrow natural levees along minor streams which have flowed in the abandoned Mississippi channels.

c) Corresponding to the above different ages, the backswamp deposits of the Maringonin-stage were first laid down beyond its natural levees over the recent substratum in the alluvial valley, then that of the Teche-stage similarly laid down between the west-side Pleistocene uplands and the Maringonin-ridge, then that of the Lafourche-stage in like manner laid down between the Maringonin-ridge and the east-side Pleistocene uplands, and finally that of the Modern-Mississippi-stage laid down between the east-side Pleistocene uplands and the Lafourche-ridge south of Donaldsonville and the older ridge north of it during its first 350 years. After conditions for the formation of the Atchafalaya Distributary were established about the year 1,500 A.D., the major portion of the Atchafalaya Basin has become the backswamp of the Modern Mississippi on its west side.

d) Contemporary with the above different stages, the different major deltaic deposits were formed seaward by the deltaic distributaries of each stage. The authors have ably contributed accounts for "Deltaic Plain Deposits" and "Deltaic Environments of Deposition."

e) During the Lafourche-Mississippi-Stage, in about 900 A.D., the Atchafalaya Basin was isolated by the coalescing of the Teche and Lafourche ridges in the area south of Houma, Louisiana. The southern portion of the basin was marked by a small lake area. By 1,400 A.D., when the Modern Mississippi stage had established its new course, the ancient Grand Lake of the south Atchafalaya Basin lake area reached its maximum extent to as far north as the latitude of Baton Rouge, Louisiana. Since the formation of the Atchafalaya Distributary of the Mississippi about the year 1,500 A.D., the southern basin lake area has continually deteriorated as Atchafalaya River delta growth has proceeded into the lake area.

In the northern portion of the large ancestral Grand Lake, a blanket of deltaic deposits has been laid down in three layers consisting of two bottom layers of ancient deltaic deposits and a top layer of modern Atchafalaya deltaic deposits. The lower bottom layer of red color deposits were derived from Red River sediments and the overlying brown and gray-brown deposits were mixed Red River and Mississippi River deposits. Overland deltaic deposits have been found to -21 feet below M.G.L. from a boring located west of Chicot Pass of the Atchafalaya Distributary.

The main channel of the Atchafalaya River is leveed above Krotz Springs, Louisiana, below which under natural conditions the main channel separates

into numerous small, shallow streams which empty in a typical deltaic pattern into the Grand Lake. As a result of the filling of the ancient Grand Lake by the Atchafalaya deltaic deposits, the northern boundary of the present Grand Lake has continually receded southward.

Composite Group of Deposits

The authors treat of at length the deposits which fill the entrenched valley of the Mississippi and make up its alluvial plain, and mention some of the engineering problems associated with each group. The authors' treatment of the associated engineering problems is in each group singularly for that group. In general, except for foundations carrying rather light structures, it is to be expected to encounter, in the lower alluvial valley, a composite group of recent topstratum deposits of fine-grained materials to the recent substratum of fine, medium, and coarse sand. In the region adjacent to north of the present Grand Lake, for instance, the engineer may encounter a sequence of successive underlying layers consisting of (1) overland deltaic deposits as the top layer, (2) backswamp deposits of comparatively recent origin, (3) buried ancient natural levee deposits of the Maringonin-Stage, (4) point bar deposits of the Maringonin-Stage, and (5) ancient braided stream deposits. For heavy constructions, the substructure will have to penetrate the first four groups and part of the fifth and found on the denser part of the fifth group, though part of the top deltaic deposits may have to be excavated.

In the lower alluvial valley, such more complex engineering problems are by far the more common for heavy constructions than the single-group-deposit foundation materials treated by the authors.

The Delicate Situation of the Present Lower Mississippi

Under "River Migration and Channel Diversion," the authors emphatically state: "Of considerable importance to engineers entrusted with site location along the banks of the Mississippi River is a valid prediction of the direction the river may migrate during the life of an installation." The stability of a river regimen depends on the complex interaction of a number of separate factors, such as valley slope, load, nature of the bed and bank materials, velocity and volume of discharge, and in turn affects course migration. In the meandering pattern of the Mississippi which has characterized its course since sea level became stationary, the orderly growth of bends, together with a knowledge of bed and bank materials and accurately mapped historic bank lines, make it possible to predict future behavior.

Both straight reaches and bends may not be stable. A straight reach may be adversely affected by the gradual channel migration of upper and lower bends exposed to two contemporaneous actions of the river; bank caving on the outside of the bend or concave bank, and bar building on the inside of the bend or convex bank. As the bends enlarge themselves, the tail of the upper bend and the head of the lower bend will approach each other. The channel shoals between the pools of the bends constitute the crossing areas where velocities are highly variable and where the position of the thalweg changes constantly as bar deposits encroach from both ends. Aggradation and degradation do occur locally and, in fact, must occur whenever the stream channel migrates. River channel migration, however, can be effectively

controlled by either indirect or direct measures or both, such as well planned groins and bank revetment, when encroaching signs have become noticeable.

Fortunately, after careful study of engineering records and channel characteristics of the Mississippi River, Gerard H. Matthes concluded that the stream exhibits no over-all tendency either to aggrade or degrade its channel and for this reason he termed it "poised." A poised stream is, however, in a condition of very sensitive equilibrium, and if there is any change in the controlling factors, the equilibrium is shifted in a direction that tends to absorb the effect of the change. The adjustments thus ensued may rapidly propogate upstream or downstream, or both, from the location where such a change has taken place or been introduced. A major cut-off may be the cause of a minor change. An increasing or decreasing in discharge or load may be the cause of a major change. The threatened capture of the Mississippi by the Atchafalaya Distributary has been thought to have accelerated by downcutting of the Atchafalaya channel as the discharge increases and silting of the Mississippi channel as the discharge decreases.

Be it more or less poised as the Mississippi is now, the increasing diversion of the Atchafalaya Distributary continues to threaten the poised condition of its master stream. According to investigations made by the Waterways Experiment Station on the problem of Mississippi River Diversion, it has been conclusively shown that a conservative maximum estimate is placed at 40 percent diversion of the Mississippi flow in order to maintain the present course of the master stream below Old River junction. Both the trend established by the percentage of total annual flow diverted from the Mississippi through Old River and trends based on the establishment of a channel from the head of the Atchafalaya Distributary to its mouth, show that 40 percent of the Mississippi flow will be diverted around 1970. It is a threatening that will affect the present course of the lower Mississippi not only as a drainage channel but also as an artery of water-borne commerce.

The Geophysical Dynamic Complexity of the Atchafalaya Distributary as a Geologic Factor in the Lower Alluvial Valley

The Atchafalaya Basin is situated entirely within the boundary walls of the lower Mississippi alluvial valley, and, to be more exact, between the former Teche-Mississippi and the Modern-Lafourche-Mississippi. It lies over the buried ancient Maringonin-Mississippi ridge, the backswamps of the Maringonin-, Teche-, Lafourche-, and Modern-Mississippi stages, the overland deltaic deposits of Red River, mixed Red River and Mississippi, and Atchafalaya Distributary. Not only is the Atchafalaya Basin the major part of the lower region of the lower Mississippi alluvial valley, but also the Atchafalaya Distributary constitutes the most dynamic, geologic force system in the lower alluvial valley during recent centuries with ever increasing importance. The above physical environments coupled with the ever increasing discharge through this distributary of already up to 30 percent of the total annual flow of the Mississippi past the latitude of Old River junction, certainly deserve their inclusion in the geological treatment of the lower Mississippi alluvial valley.

The geophysical dynamic forces with which the Atchafalaya Distributary is playing are of varying complexity from its head to the Grand Lake-Six Mile Lake system.

The Increasing Discharge

The formation of the Atchafalaya distributary was conditioned as far back as about 1500 A.D. A map dated 1578 A.D. drawn by Monk Ptolemy, who accompanied DeSoto's expedition in 1542, shows conclusively that the Atchafalaya then, as now, served as an outlet for the Mississippi. After the rafts of logs had been entirely removed before 1855, the distributary has progressively increased its flow which has been diverted from the Mississippi. Since 1900, the discharge of the Mississippi through this distributary has increased from 5 to 25 percent of the total annual flow, and from 15 to 30 percent of the total annual flow past the latitude of Old River. The estimated limiting condition of conservative maximum diversion of not more than 40 percent in order to keep the present course of the lower Mississippi is ever approaching.

This tendency of increasing discharge is further facilitated by the topographical advantage that the Atchafalaya Distributary has a much shorter distance and much steeper slope from the Old River junction to the Gulf than the master stream to its present delta mouth.

The Enlargement of the Leveed Segment

The increase in discharge through the Atchafalaya Distributary has resulted in a corresponding increase in channel width together with a gradual increase in depth through the years. North of Krotz Springs, Louisiana, the Atchafalaya channel is leveed. In some areas of this leveed segment, channel width has doubled that of 50 years ago. This trend of enlargement magnifies itself along many reaches of the river by caving of opposite banks, a condition not observed on the Mississippi River where caving along an individual reach is usually confined to a single bank. This gradual enlargement of the cross-sectional areas of the leveed segment supplies material for natural levee formation in the unleveed segment below Krotz Springs, Louisiana.

The Forming of Natural Levees

Along the lower unleveed segment of the Atchafalaya, natural levees are forming at a rapid rate, and it is conceivable that they will eventually become larger in areal extent than those along the artifically controlled upper reaches. Along the unleveed segment, the silty sand and fine sand carried in suspension during overbank flow are deposited over the above bank surface as the velocity slackens. This process of building up natural levees will continue to be pronounced so long as the cross-sectional areas of the leveed segment gradually enlarge to increase the load of the discharge.

The Building Up of Overland Deltaic Deposits

The retreating of the northern boundary of the Grand Lake southward has been accounted for in an earlier passage. From 1932 to 1950, the growth in thickness of deltaic deposits in the northern portion of the ancient Grand Lake varies generally from below 3 feet to a maximum of about 10 feet.

The Deepening of the Unleveed Segment

The Atchafalaya lower main channel now has developed to a varying depth from 40 to 80 feet. This lower, unleveed segment of the Atchafalaya main

channel consists principally of thick backswamp clays which offer most resistance to scour and consequent cross-sectional enlargement. Hence the rate of increase in channel widths and depths in the unleveed segment of the main channel will be much slower than in the upper leveed segment. However, as natural levees are built up, and as volume of discharge continues to increase, heavier scour will result in the channel. Except for minor channel migration which might be initiated by local high substratum sand areas, the Atchafalaya main channel, south of the leveed segment, should develop a deep, narrow, nearly straight channel, comparable to the present Mississippi south of Donaldsonville, Louisiana.

Single or Multiple Channels

In the unleveed segment, during flood and high-water stages, considerable volumes of water are carried along subsidiary channels, notably the Bayou L'Embarras—Lake Rond channel which has carried as much as 60 percent of the total discharge during recent floods. Hence, the question arises in the probable location of the future main channel, whether by enlargement of the present main channel on the east or, instead, the L'Embarras—Lake Rond Channel on the west, or the coexistence of multiple channels.

The Repeating of the Deportment of Characteristic Caving

The deportment of characteristic caving of opposite banks of the upper segment of the Atchafalaya Distributary may be repeated in the future, though most probably to a less extent, in the unleveed segment of the main channel, when the latter has attained enough capacity and before it reaches a state of equilibrium.

The Developing Pattern of Capturing the Master Stream

The Atchafalaya Distributary is following a developmental pattern so characteristic of former Mississippi River diversions that each of which within an estimated period of 100 or more years captured the entire flow of the master stream. Now that a channel capable of carrying low-water flow has been established and enlargement has begun, no natural processes are known which in the case of the Atchafalaya might prevent further enlargement and eventual diversion of the entire Mississippi flow, until the Corps of Engineers have completed their positive artificial control. Even after the completion of such large engineering control works, there is no eternal guarantee that a major breach would not happen, should untenable conditions due to climatic abnormalties occur, for the control works are located in an alluvial plain.

The above are the geological dynamic forces at work, their consequences, and their tendencies, and hence their engineering significance in the planning and location or any major engineering project in relation to trans-basin transportation routes, or control and regulation of the Atchafalaya Distributary.

Past Foundation Lessons in the Lower Alluvial Valley

The authors have cited some past foundation lessons in the lower-left region of the lower alluvial valley. It is attempted in what follows to present some past foundation case lessons occurred in the lower-right region.

a) The Shifting of Thalweg

The abandoned bridge structure of the Texas and Pacific Railroad over the Old River at Torras, Louisiana, has pier foundations down to -10.08 feet near the south bank and to -101.92 feet near the center of the river. The M.L.W. El. and M.H.W. El. are respectively at +8.2 feet and +49.6 feet. Though it had carried no traffic after abandonment, the north span and approach collapsed in 1948, showing that foundations were not built to sufficient depths to meet the possible shifting of the thalweg. At this crossing the thalweg had shifted itself from south to north between 1917 and 1951 for a distance of more than 900 feet. Had it not been for the abandonment of the bridge, groins of proper location, direction, and dimensions should have been built to keep the thalweg unchanged as soon as signs of shifting had been noticed.

b) Provisions for Thalweg Shifting

The bridge for U. S. Highway No. 190 and No. 71 over the Atchafalaya Distributary at Krotz Springs, Louisiana, has all its four river piers founded to the same depth of -139.95 feet. The M.L.W. El. at this crossing is at +6.1 feet, while the M.H.W. El. is at +33.0 feet. The founding of all the river piers to the same depth is a resource that will permit whatever shifting of the thalweg that may happen.

c) The Deepening of Channel

The Kansas City Southern Railroad and the Louisiana State Highway No. 30 have a combined bridge over the Atchafalaya Distributary at Simmesport, Louisiana. It was a wise precaution to build the main river piers down to -180 feet for both the old structure of 1928 and the modification of the structure in 1938. Here the elevations of M.L.W. and M.H.W. are respectively at +7.3 and +46.4 feet. The channel between piers No. 2 and No. 3 had a maximum increase in depth of 44 feet from 1888 to 1928.

d) Provisions for Channel Deepening

The Missouri Pacific Railroad has a bridge over the Atchafalaya Distributary at Krotz Springs, Louisiana, with its old structure piers built to depths from -67.01 to -89.43 feet. The Atchafalaya River has considerably deepened its channel in this segment. Consequently the 1941 modification of the structure has river piers founded to -140.0 feet, constituting an additional provision of more than 50 feet for channel deepening as compared with the older piers.

e) The Reward for Shallow Bridge Foundation

The Missouri, Louisiana, and Texas Railroad Bridge over the Atchafalaya Distributary at Atchafalaya, Louisiana, had its river piers founded only to -83.6 feet and -84.08 feet. The M.L.W. El. is at +5.8 feet, while the M.H.W. El. reaches 20 feet higher. The pivot pier of the swing span was overturned in June, 1928; all spans were removed in November, 1929; and the east rest-pier was destroyed in 1935. Such was the reward for inadequate pier foundation depths in the deepening channel of the distributary in the lower alluvial valley.

Commendation and Acknowledgment

The authors are to be highly commended for their most timely and worthy contribution to the advancement of human knowledge on "Mississippi Valley Geology—Its Engineering Significance," especially the lower Mississippi valley to which their paper is devoted. The importance of this subject cannot

be overestimated. The authors' treatment of the various environments of alluvial deposition and the resulting soil formations is elucidating, and their relation to engineering problems is suggestive to even further inquiry, conception, and correlation.

The writer's discussion has mainly focused on what appears to the authors as forming a subject by itself or in areas previously covered, but, nevertheless, essential to the lower Mississippi valley geology. It has been attempted to bring forth light in this discussion from pertinent published and unpublished information, available to the writer, on the subject in a concise manner to help comprehend the depth and breadth of the issue the authors have brought to the fore.

The writer wishes to acknowledge his indebtedness to all previous authors who have made contributions relevant to the subject. The multitudinous and interwoven character of the sources of information with respect to the writer's discussion would make it a heavy burden on him if they were all footnoted as he should. The writer is especially indebted to Professor Harold N. Fisk for his original contributions on the subject, from which the writer has drawn some of the established geological concepts of the region, to the published reports of the Mississippi River Commission, and to the U. S. Waterways Experiment Station. Appreciation is due to all the authors of the following references.

REFERENCES

1. Elliot, D. O. The Improvement of the Lower Mississippi River for Flood Control and Navigation, May, 1932, Waterways Experiment Station, Corps of Engineers, U.S. Army, Vicksburg, Mississippi.

2. Russell, R. J. and Russell, R. D. Mississippi River Delta Sedimentation, Recent Marine Deposits, American Association of Petroleum Geologists, Tulsa, 1939.

3. Matthes, Gerard H. Basic Aspects of Stream Meanders, Transactions, American Geophysical Union, Part III, 1941.

4. Fisk, H. N. Geological Investigations of the Alluvial Valley of the Lower Mississippi River, December, 1944, Mississippi River Commission, Vicksburg, Mississippi.

5. Friedkin, J. F. A Laboratory Study of the Meandering of Alluvial Rivers, 1945, Waterways Experiment Station, Corps of Engineers, U. S. Army, Vicksburg, Mississippi.

6. Fisk, H. N. Fine-Grained Alluvial Deposits and their Effects on Mississippi River Activity, 1947, Waterways Experiment Station, Corps of Engineers, U. S. Army, Vicksburg, Mississippi.

7. Mackin, J. Hoover Concept of the Graded River, Bulletin, Geological Society of America, Vol. 59, 1948.

8. Matthes, Gerard H. Mississippi River Cutoffs, Transactions, American Society of Civil Engineers, Vol. 113, 1948.

9. Turnbull, W. J., Krinitzsky, E. L., and Johnson, S. J. Sedimentary Geology of the Alluvial Valley of the Lower Mississippi River and Its Influence on Foundation Problems, Applied Sedimentation, John Wiley & Sons, New York, 1950.

10. The Atchafalaya River Study, Vol. 2, May, 1951, Mississippi River Commission, Vicksburg, Mississippi.

11. Fisk, H. N. et al. Geological Investigation of the Atchafalaya Basin and the Problem of Mississippi River Diversion, Vol. 1, April, 1952, prepared for the President, Mississippi River Commission, by the Waterways Experiment Station, Corps of Engineers, U. S. Army, Vicksburg, Miss.

12. Fisk, H. N. et al. Geological Investigation of the Atchafalaya Basin and the Problem of Mississippi River Diversion, Vol. II, April, 1952, prepared for the President, Mississippi River Commission, by the Waterways Experiment Station, Corps of Engineers, U. S. Army, Vicksburg, Miss.

13. Hardin, John R. Mississippi-Atchafalaya Diversion Problem, Military Engineer, Vol. XLVI, No. 310, March-April, 1954.

14. Li, S. T. Engineering Geology Along the Acadian Thruway, (Unpublished) Technical Memorandum No. 8, July, 1956, to Palmer and Baker, Inc., Consulting Engineers, Mobile, Alabama.

15. Li, S. T. Past Subsurface Explorations and Foundation Depths of Existing Bridges Relevant to the Acadian Thruway Transbasin Territory, (Unpublished) Technical Memorandum No. 9, July, 1956, to Palmer and Baker, Inc., Consulting Engineers, Mobile, Alabama.

DIVISION ACTIVITIES
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

NEWSLETTER

July, 1957

DIVISION EXECUTIVE COMMITTEE
October, 1956 - October, 1957

		Contact member for committees on
R. B. Peck	113 Talbot Lab. Univ. of Illinois Urbana, Illinois	Eng'g. Geology
S. D. Wilson	Shannon & Wilson 2208 Market Street Seattle 7, Washington	Earth Dams; Road & Airfield Soil Problems
R. E. Fadum	Dept. of Civil Eng'g. North Carolina State Coll. Raleigh, N. C.	Glossary; Technical Sessions
S. J. Johnson	Moran, Proctor, Mueser and Rutledge 420 Lexington Avenue New York, New York	Publications Grouting
J. O. Osterberg (Secretary)	Dept. of Civil Eng'g. Northwestern University Evanston, Illinois	

Note: No. 1957-13 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers, Vol. 83, SM 3, July, 1957.

Donald Wood Taylor, M. ASCE

Died December 24, 1955

Donald Wood Taylor was born in Worcester, Massachusetts on December 2, 1900, the son of Walter B. and Clara (Wood) Taylor. He attended the Worcester Public Schools and was graduated from North High School in 1918. He received his undergraduate engineering education at Worcester Polytechnic Institute where he graduated in 1922 with a Bachelor of Science degree. He was a member of Sigma Phi Epsilon fraternity.

After graduation, Mr. Taylor joined the U. S. Coast and Geodetic Survey and was assigned to mapping and offshore sounding along the coast of Alaska and the Aleutian Islands. He then worked as an engineer for the City of Los Angeles, the Edward Miner Construction Company and the New England Power Company. During the latter engagement, in 1928-29, he was assigned to a cooperative research project with the Soil Mechanics Laboratory of the Department of Civil and Sanitary Engineering at the Massachusetts Institute of Technology. His accomplishments were so outstanding that when a staff research post at M.I.T. became available in 1932 it was offered to him. He accepted this position and thereby started an illustrious professional career in Soil Mechanics. He took graduate courses simultaneously and was awarded the degree of Master of Science in Civil Engineering in 1942.

In 1938, he was appointed Assistant Professor of Soil Mechanics and in 1944 he was advanced to Associate Professor. From 1939 until his death, Professor Taylor was Head of the Soil Mechanics Division at M.I.T. and in charge of teaching and research of this division. He rapidly became one of the world's leading authorities in the field of Soil Mechanics, and his textbook "Fundamentals of Soil Mechanics," published in 1948, is of a type all too rarely seen. In 700 pages, it compresses all the basic concepts of the science up to that time so that it is an invaluable aid for reference as well as for teaching. It is widely used, and will undoubtedly hold its preeminent position for many years to come.

In addition to a heavy teaching schedule, Professor Taylor performed and directed extensive institutional and sponsored research in Soil Mechanics, and he conducted an active practice as a consultant to governmental and private organizations. He served on numerous consulting boards for the Corps of Engineers, U. S. Army, dealing with earth dams, flood walls, test tunnels, many theoretical and experimental studies at the Waterways Experiment Station, and several classified projects associated with atomic blast effects and the behavior of soils under dynamic loading. He was a consultant on earth dams being built by the New England Electric System and the Southern California Edison Company, and he also served as a consultant for Fay, Spofford and Thorndike, Jackson and Moreland, Charles H. Maguire and Associates on foundation problems.

Among the earth dam projects for which Professor Taylor was a consultant were the Union Falls Dam for the Central Maine Power Company, and the Littleton Dam Development for the New England Electric System. From 1946, he was an advisor to the Engineering Staff of the Southern California Edison

Company and consulted on the design of the Vermilion Dam and the Mamoth Pool Dam, both in the Sierra Nevadas of California.

During the last fifteen years of his life, Professor Taylor was a consultant to Fay, Spofford and Thorndike in connection with a great variety of foundation and earth pressure problems. His advice and recommendations assisted greatly in the solution of design problems on airfield runways, highway embankments, deep water bulkheads, pile foundations, consolidation of compressible material by sand drains and other means. The projects involved were widely scattered in type and location and included the South Boston Army Supply Base, East Boston Pier No. 1, the Gulf Oil Terminal in Chelsea, Massachusetts, Air Base in Greenland, Portland-South Portland Bridge in Maine, South Boston Dry Dock No. 4, Garden State Parkway, New Jersey Turnpike and the Massachusetts Turnpike. Commenting on these contributions, Mr. Frank L. Lincoln of this firm wrote:

> "In every case his analysis, judgement and conclusions were proven sound by the successful accomplishment of the constructions. His cooperative and friendly manner were most helpful in carrying on the work, and he enjoyed the confidence of his associates in his engineering ability."

In the four years preceding his death, Professor Taylor was called in frequently on problems connected with the John F.Fitzgerald Expressway. This is an expressway through downtown Boston, partly elevated and partly in a subway structure, with a construction cost of some $70,000,000 for the portions in which Professor Taylor was involved. There were many problems, two of which deserve special mention: First, the design of an earth fill over weak silt on the shore of the Charles River, which is note-worthy because the work was accomplished without the use of a bulkhead or other costly retaining structure; and second, the stabilization, in another area, of embankments over extreme depths of soft clay and the transition structure from these embankments to solidly-founded viaducts. In the latter case it was decided to place the viaduct abutments on new fill without piles, the compressible soils under the abutments having been pre-consolidated to some extent by overloading and in some cases by the use of vertical sand drains. The bold recommendation for this procedure was accepted by the clients as a result of their confidence in Professor Taylor's judgment, and the work was placed under contract after his death.

Professor Taylor's work constitutes an outstanding contribution to the advancement of both basic knowledge and practical application of Soil Mechanics. His book, a leading text in its field, is used throughout the world. His papers published in the technical journals represent only a part of his work, since the results of many of his investigations are contained in unpublished or classified reports.

He is perhaps best known for his work on the stability of slopes. His paper on "Stability of Earth Slopes" was awarded the Desmond Fitzgerald Medal in 1938, the highest award of the Boston Society of Civil Engineers, and some of the concepts and diagrams of this paper are incorporated in nearly all leading texts on the subject. Many of his papers and reports deal with strength and consolidation characteristics of soils; they contain new and interesting concepts, and his pioneer work on measurement of porewater pressures during strength tests is of primary importance. His theory for the action of pressure cells in soils and his development of a cell for the measurement of

shearing stresses is a major contribution to the solution of the difficult problem of reliable measurement of stresses in and exerted by soils.

Professor Taylor was a member of and active in the work of many professional organizations. He was a member of the Boston Society of Civil Engineers and for many years, the chairman of its Committee on Subsoils of Boston. At the time of his death, he was Vice President of this society and had been nominated for its presidency. He was a member of the American Society of Civil Engineers and several of its technical committees, the American Society for Engineering Education, the Highway Research Board, and the International Society of Soil Mechanics and Foundation Engineering. He served as Executive Secretary of the latter society from 1948 to 1953; and attended conferences in Holland in 1948, and in Switzerland in 1953, and was a delegate to the UNESCO Conference in Paris in 1950. He was a member of the Executive Committee of the U. S. National Council on Soil Mechanics and Foundations Engineering.

He was also a member of the honorary fraternities Sigma Xi and Chi Epsilon.

Impressive though this record is, it does not tell a fraction of the story of Professor Taylor's personal contribution to the engineering profession and to society. He was essentially a modest man, quiet, gentle and unassuming with a pleasant, kindly, friendly manner. His many friends, former students and colleagues will always remember with admiration the all-too-rare combination of sterling qualities which typified him: Sincerity, gentle friendliness, intellectual honesty and integrity.

No doubt one of his greatest contributions was the influence he had on the young men who studied or worked under his supervision. To these the technical knowledge and education he imparted was important, but of even greater importance was the friendly encouragement, inspiration and example which he contributed. He will always live in their memory with honor and affection.

Throughout his career, his work, whether in teaching, or in research, or in engineering practice, was characterized by the highest professional standards, by great care and attention to accuracy, and by all allegiance to the scientific method in his quest for new knowledge. His high standards were as evident in his personal life as they were in his professional career. A man of utmost integrity, he was guided in his thinking and his actions by an analysis of existing facts and by a desire to meet problems and to treat people with fairness and full cooperation.

Those who through shared interests and activities have had the pleasure and privilege of knowing Donald Wood Taylor will miss a dedicated colleague and a friend held in high esteem.

On October 13, 1928, he was married to Beulah Elizabeth Nyman of Marlborough, Massachusetts. He is survived by his wife, his parents, and one sister, Mrs. Norman M. French of Worcester.

The Soil Mechanics and Foundation Division is indebted to Professor C. H. Norris of M.I.T. for the above memoir.

LOCAL SECTION NEWS

Activity in the San Francisco Area

The Scil Mechanics Division of the San Francisco Section has continued with its active program. On the 28th of March a meeting was held at the

University of California in Berkeley at which Mr. Wesley G. Holtz, Chief, Earth Materials Laboratory, U. S. Bureau of Reclamation, Denver, gave an interesting talk on "Thick Compacted Earth Linings for Canals." Mr. Holtz discussed the various types of linings generally used for canals with specific emphasis on thick compacted earth linings. He pointed out that the advantages of the thick linings over thin linings were that they were generally more impervious, there was less chance of damage by animals and plants, and they were generally more economical to build than other types of lining because it is not necessary to compact the entire embankment beneath an earth lining as in the case of embankments beneath concrete linings. Experience in the Bureau of Reclamation on hundreds of miles of canals shows that the best soil for canal linings is a well-graded gravel with a clay binder. This material is most resistant to erosion, is impervious, compacts well and has less chance of loss of strength or change in volume with changes in moisture content.

The next meeting of the local division is scheduled for May 15th at the University of California Faculty Club at 6:15 p.m. The technical program will begin at 7:30 in Room 204 of the Engineering Building. Speaker for the evening will be Dr. D. H. Trollope, Senior Lecturer in Civil Engineering at the University of Melbourne. Dr. Trollope will talk on "Soil Mechanics Problems in Australia."

Pittsburgh Soils Conference Well Attended

The six session conference sponsored by the Pittsburgh, Pennsylvania local section committee on Soil Mechanics and Foundation received the enthusiastic support of contractors, engineers, and architects in the area. Between 250 and 300 men attended each of the meetings. It was a good example of the way in which a local section committee can foster better understanding of the divisions work to those in associated fields. Have you done anything in your own area along these lines? Try it. You may be surprised at the result.

AERIAO

At the Society Convention in Jackson, Mississippi this last February Dr. Jerry Leonards of Purdue described an interesting sand he encountered in the Azores. We asked him to give us the following story to pass on to you.

Santa Maria, Azores, is a volcanic island that has erupted a number of times in the past. So far as is known igneous activity is now dormant. At one stage in its geologic history the island was depressed below sea level, and strata of gravel and boulders (later cemented to conglomerate), sandstone, and limestone were successively deposited over the older lava. These strata were subsequently uplifted above sea level and subjected to erosion and weathering. Later eruptions spewed lava over these sedimentary deposits, or over the surfaces from which they had been eroded, which in some instances is the surface of the older basalt. During this period, molten rock intruded into cracks and fissures, particularly through the older rock, forming irregular dikes of fine-grained but highly resistant igneous rocks. At the present time, only small remnants of the sedimentary deposits remain, which generally are a few feet or less in thickness.

The base rock of the lower plateau, where the air base is located, is an olivine basalt that appears to have originated from rapid cooling of a silica deficient lava. The paucity of crystallites in the volcanic glass groundmass,

and the nature of the rock in situ, is suggestive of submarine formation. The rock consists essentially of interlocking grains of augite set in clear brown glass containing phenocrysts of olivine that have been altered along their borders, and places along the partings between grains, to brown iddingsite. Typical physical properties of the rock are tabulated below:

Unit Weight	Hardness	Compressive Strength	Young's Modulus
lbs./cu.ft.	Moh Scale	psi x 10^3	psi x 10^6
182	7 +	30	4.5

Soil cover on the island is extremely thin. Torrential rains cause severe erosion and in the hills sufficient soil for farming can be obtained only by constructing a patchwork of stone fences to trap the run-off and cause it to deposit its load. Where present, the soil is residual, having been derived principally from the olivine basalt; minor amounts of residual limestone soil are also in evidence. These soils are highly plastic, sticky, and very low in stability in their natural state. A small area in the vicinity of Prainha Bay has beach sand, but it exists as a thin veneer and is present for only a few months of the year.

The most interesting materials on the island were deposits of fractured, weathered, olivine basalt known locally as "areiao"—translated, coarse sand. It consists of small clasts (pieces) of olivine basalt occurring in a weathered matrix. The degree of decomposition of the clasts, and the amount of weathered matrix present, varies widely from deposit to deposit and at different locations in the same deposit. Deposits of essentially unweathered clasts with low percentages of weathered matrix have the appearance of openwork pea gravel. No significant difference was found in the amount of olivine present in clasts from deposits having high and low percentages of weathered matrix; furthermore, the olivine in the clasts showed somewhat less alteration than that in the intact basalt. It appears that the presence of areiao is associated with closely spaced fractures developed as a result of flow patterns in the lava, rather than with differences in the amount of olivine present.

Once exposed, areiao is readily eroded; accordingly, deposits may be found only where they are capped by sedimentary deposits, by later lava flows, or wherever they have been contained by patchworks of intrusive dikes developed during subsequent igneous action. These areiao deposits are of great economic importance on the island being the only material available, which can be obtained by normal borrow operations, that is acceptable for the construction of bases and sub-bases for road and airfield pavements, and for use as a road metal. It is very difficult and expensive to crush the basalt.

CONVENTIONS AND CONFERENCES

World Conference on Prestressed Concrete

There is to be a World Conference on Prestressed Concrete presented by the Department of Engineering and University Extension of the University of California from July 29 through August 2, 1957 in San Francisco. Of particular interest to division members are the afternoon Sessions on August 1. The subject for the afternoon Session is Prestressed Pavements, Wharves, Piles and Dams. The following papers will be presented:

Prestressed Airfield Runways - F. Mellinger, Director, Ohio River Laboratories, U. S. Corps of Engineers, Ohio

Prestressed Piles and Wharf Members - Ben C. Gerwick, Jr., President, and W. J. Talbot, Chief Engineer, Ben C. Gerwick, Inc., California

Prestressed Concrete Cylinder Piles - M. Fornerod, Chief Engineer, Raymond Concrete Pile Company, New York

Prestressed Dam in Algeria - Jean Muller, Freyssinet Company, France

Further information can be obtained by writing T. Y. Lin, Professor of Civil Engineering, University of California, Berkeley 4, California. Proceedings of this conference will be available for $10 per copy.

PUBLICATIONS

Soil Mechanics Bibliography (1920 - 1946)

In 1950 the Institution of Civil Engineers published a bibliography on Soil Mechanics which covered the period from 1920 to 1946. This volume has been out of print for some years but occasional requests for a copy have caused the Institution to consider printing more copies. However, before proceeding, it has been decided to try to determine the demand. The price will be £5 per copy. Division members who would be interested in obtaining a copy should notify the Secretary, The Institution of Civil Engineers, Great George Street, London S. W. I. Requests for copies should be sent by May 31, 1957, if possible.

London Conference Proceedings

Members of the International Society of Soil Mechanics and Foundations Engineering who are unable to attend the Conference and desire copies of the Proceedings should complete Form R but write on it "non-attending Member," and send the form, with a remittance for £17.10s.Od. per set of 3 volumes, to the Secretary of the Conference. Forms received before the end of May will qualify for delivery at the same time as Registrants. Forms received later will be dealt with as soon as possible, but under no circumstances will non-attending Membership be continued after 21 August 1957, and after this date requests for copies of the Proceedings will need to be sent to the Publisher:-

Butterworths Scientific Publications Ltd., 4/5 Bell Yard, Temple Bar, London, W. C. 2.

OCTOBER NEWSLETTER

Deadline date for arrival at this office of contributions for the April Newsletter: August 20, please.

Bernard B. Gordon, Assistant Editor
Porter, Urquhart
1140 Howard Street
San Francisco 3, California

Alfred C. Ackenheil, Editor
Department of Civil Engineering
University of Pittsburgh
Pittsburgh, Pennsylvania

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

VOLUME 82 (1956)

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c]; 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

VOLUME 83 (1957)

JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246 (HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303 (ST4), 1304(ST4), 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318 (ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

c. Discussion of several papers, grouped by Divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS